A STUDY ON PROVISIONS OF INDIAN STANDARDS FOR ASEISMIC DESIGN OF BUILDINGS

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by
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CERTIFICATE

It is certified that this work "A STUDY ON PROVISIONS OF INDIAN STANDARDS FOR ASEISMIC DESIGN OF BUILDINGS" by Anil Kumar Patnaik has been carried out under my supervision and that this work has not been submitted elsewhere for a degree.

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TABLE OF CONTENTS

ABSTRACT

This thesis has been organized in three parts. In Part I, ductility provisions of IS codes have been thoroughly reviewed and compared with those in American codes. The provisions needing revisions, inclusions or clarifications have been identified. Suggestions made include rationalization of performance factor, providing of ductility detailing as per zones rather than α_h, introduction of strong column - weak girder concept in design, revision of minimum reinforcement requirements, etc. A method has also been suggested for determination of plastic moment capacity for R.C. beams.

In Part II, the effect of providing ordinary and ductile concrete frames on the cost of buildings has been investigated for different zones. Three buildings with significant difference in design features have been chosen and designed for ductile and non-ductile construction. The quantity of steel and cost of skeletal frames have been evaluated for these buildings and premium for providing earthquake resistance calculated. The introduction of performance factor has increased cost of structures not detailed for ductile construction in zones III to V. It is economical to use ductile concrete frames in zones III to V, however in zones I and II providing ductile concrete frames is uneconomical.

In Part III, design forces obtained by seismic coefficient and response spectrum methods have been compared for the three buildings. The design forces resulting from response spectrum method are significantly lower than those obtained by seismic coefficient method because the fundamental time periods resulting

from dynamic analysis of the buildings are much higher than those evaluated by the code specified approximate formula for seismic coefficient method. This formula needs to be rationalized so that the protection from earthquakes provided to the buildings will be reasonably same by both the methods.

ACKNOWLEDGEMENT

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ANIL KUMAR PATNAIK

PART I

DUCTILITY REQUIREMENTS IN INDIAN CODES FOR ASEISMIC DESIGN OF R.C FRAME STRUCTURES: A REVIEW

Earthquake resistant design involves determination of expected seismic forces and designing the structural members to resist these forces. Bureau of Indian Standards has published two codes, IS:1893 - 1984 (Ref.1), which is primarily a load standard, specifying minimum seismic design loads for structures and, IS:4326 - 1976 (Ref.2), which contains design standards, setting down requirements by which to proportion and detail members.

The seismic codes do not intend to ensure that no structure shall suffer damage during a large earthquake. For instance, IS:1893 - 1984 mentions that, "It has been endeavoured to ensure that, as far as possible, structures are able to respond, without structural damage to shocks of moderate intensities and without total collapse to shocks of heavy intensities." This is because a structure which can withstand strongest ground shaking without damage will be too expensive to build. Hence, it is obvious that the non-linear behaviour of structure, i.e., beyond its yield, will greatly affect its seismic design. It is, therefore, important that structures should be more ductile for better performance during earthquakes. Ductility of a structure means capacity to deform to a large extent without loss of strength before collapse, as compared to its deformation at yield point.

Seismic codes around the world ensure adequate ductility of a structure in two ways. Firstly, design seismic forces for a ductile structure are less than those for a brittle structure. Secondly, it is required that the structures to be built in a

highly seismic zone must have a minimum level of ductility. In this Part-I, IS code provisions for ductility requirements have been thoroughly reviewed and compared with those in American codes. An attempt has been made to identify the areas which are implicit, non-existent or confusing and need to be revised or included in the future revisions of the Indian codes. Also included are the author's view points on some of the clauses needing thorough study and revision by the Earthquake Engineering Sectional Committee before bringing out the future editions of the codes.

2. DUCTILITY :

Ductility is one of the most important requirements of earthquake resistant design. As per Ref.(12) " High ductility is the ability of a building to sustain large deflections without failure or collapse." It is impractical to expect the structure to respond to a very strong ground shaking within its elastic range. Hence, the structure is allowed to go beyond its yield point due to strong ground motion. Ductility implies that the structure will sustain fairly large deflection beyond its yield before it collapses. This results in reduced earthquake forces experiencedby the structure.In thepost-yieldrange, the structure exhibits significant hysteretic damping and this energy dissipation reduces response. Besides, the yielding leads to softening of the structure which increases the time period of structure and it usually means further decrease in response.

Again, when the plastic hinges tend to develop, stresses are transferred elsewhere, to member sections whose energy capacity and absorption have not been fully utilized. Thus the whole

structure tends to offer resistance in severe emergencies and it is not limited to one weak section of a member in the elastic range Ref.(12).

Research has established (e.g. Ref.34) that it is reasonable to assume the deflections produced by a given earthquake input to be essentially the same, whether a structure responds elastically or yields significantly. If the member force - deformation relation shown in Fig.(1) is considered, the maximum deflection δ_{max} developed in the member is same regardless of its strength property. The ratio of the maximum deformation to the elastic - limit deformation is equal to the ratio of the force developed in purely elastic response to the member yield force, that is

$$\frac{\delta_{max}}{\delta_y} = \frac{f_{max}}{f_y} \qquad \ldots (1)$$

The ductility factor μ of the member is

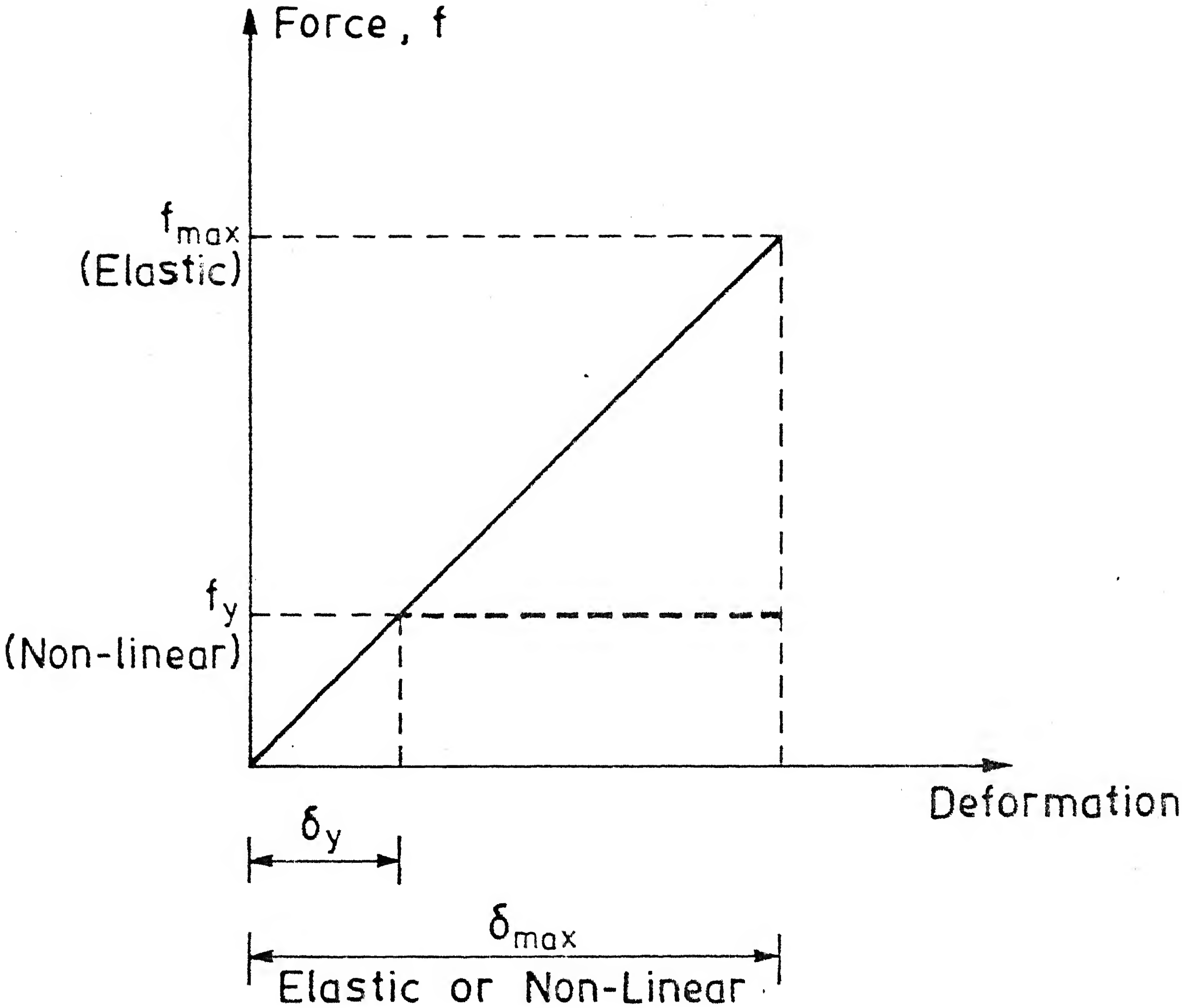
$$\mu = \frac{\delta_{max}}{\delta_y} \qquad \ldots (2)$$

i.e.,
$$f_y = \frac{1}{\mu} . f_{max} \qquad \ldots (3)$$

Equation (3) clearly demonstrates that the design forces of members are reduced with increase in the ductility factor, indicating that the ductility of the members can be advantageously used to reduce member design forces.

Ductility of the structure depends on ductility of the individual members, but there is no way to establish a direct correlation between the two. Some of the important aspects of ductility are :

(a) Axial load in members reduces ductility at column ends, the larger the axial stress, the larger the reduction (e.g., Ref.35),



Ductility Factor : $\mu = \dfrac{\delta_{max}}{\delta_y} = \dfrac{f_{max}}{f_y}$

FIGURE 1. DEFINITION OF DUCTILITY FACTOR.
(From Ref.-34)

suggesting that the plastic hinges occur at the ends of beams rather than in columns. Hence, strong column - weak girder concept of design of structures improves ductility.

(b) A flexural member exhibits large ductility before collapse if it undergoes tension failure. Thus by providing under - reinforced sections and providing limits on minimum and maximum reinforcements ductility may be improved substantially.

(c) The formation of plastic hinges involves large rotations in the members. If the failure of a member in diagonal shear is avoided before formation of plastic hinges, the member will be able to develop full curvature and will behave in a ductile manner.

(d) In reality concrete itself is not a ductile material, but if it is reinforced properly and confined by closely spaced transverse steel at proper locations, the combination will behave like a ductile material. Recognizing this, the building codes Refs. (2, 6) specify the detailing for concrete members to render them ductile.

3. DUCTILITY PROVISIONS IN INDIAN CODES :

3.1 PERFORMANCE FACTOR :

In the earlier editions of IS:1893, in design seismic force calculation, no distinction was made between ductile and brittle structures. However, the 1984 edition has introduced "Performance Factor", K, for buildings which ensures higher design forces for brittle buildings. The values of K have been specified in Table 5 of the code and reproduced in Table 1 of this Part-I. It must be mentioned that prior of 1984 edition, in the absence of K, it can be taken as 1.0. Thus, for structures

TABLE 1 : VALUES OF PERFORMANCE FACTOR, K

[From Ref.(1)]

1.No.	Structural Framing System	Values of Performance Factor, K	Remarks
1	(2)	(3)	(4)
) a)	Moment resistant frame with appropriate ductility details as given in IS:4326-1976* in reinforced concrete or steel.	1.0	-
b)	Frame as above with R.C. shear walls or steel bracing members designed for ductility	1.0	) These factors will) apply only if the) steel bracing members) and the infill panels) are taken into consi-) deration in stiffness) as well lateral stren) gth calculations pro-) vided that the frame) acting alone will be) able to resist atleas) 25 percent of the) design seismic forces
.)a)	Frame as in (i) (a) with either steel bracing members or plain or nominally reinforced concrete infill panels.	1.3	
b)	Frame as in (i) (a) in combination with masonry infills.	1.6	
i)	Reinforced concrete framed buildings [Not covered by (i) or (ii) above]	1.6	

Code of practice for earthquake resistant design and construction of buildings (first revision).

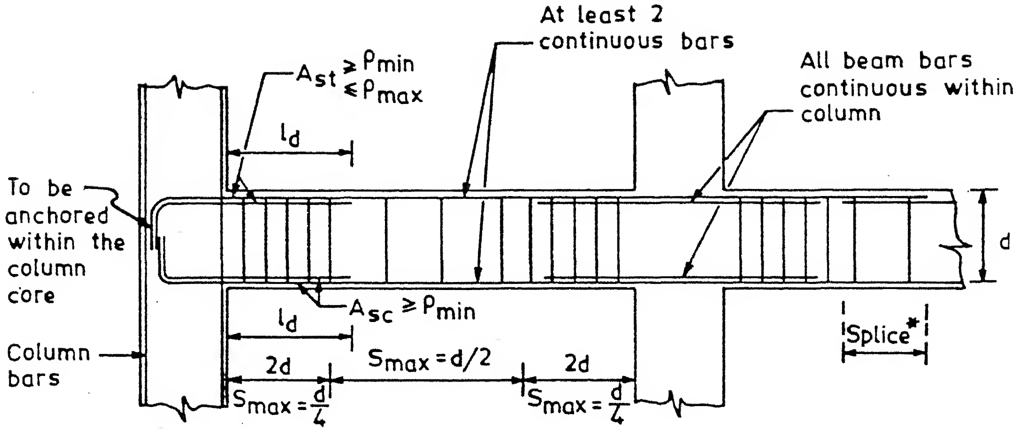
not detailed for ductility as per IS:4326, now there is an increase in seismic design force to the extent of 60 percent over what existed prior to 1984 edition, irrespective of the seismic zone.

3.2 DUCTILITY DETAILS IN REINFORCED CONCRETE CONSTRUCTION :

IS:4326 gives detailing requirements to ensure proper ductility in the structure. These provisions shall have to be adopted in all cases where design seismic coefficient α_h is 0.05 or more. The requirements of IS:4326 for flexural members have been shown in Figure 2. Similarly, the requirements of IS:4326 for columns subjected to axial load and bending are shown in Figure 3. The transverse reinforcement as required at the end of columns (Fig.3) is to be provided through beam - column joints, subjected to fifty percent reduction if the connection is confined by beams from all four sides.

4. DISCUSSION ON CODE PROVISIONS FOR DUCTILITY :

While most of the provisions in IS:4326 are similar to those of contemporary codes elsewhere, many of the recommendations are implicit and incomplete. Since the first revision of IS:4326 in 1976, a lot of research and experimental data (Refs.19 to 33) have accrued and a second revision of the code has been long felt. With the possible advent of the second revision shortly, full advantage should be taken to thoroughly revise the present form of the code to make it a self sufficient and explicitly documented code of efficient and economical practice for earthquake resistant design of structures. A well documented code could also relieve the designer of bureaucratic pressures to reduce the cost of structure and will enable him to provide sufficient protection to the structure from probable earthquake forces.



Minimum Reinforcement

M15 Concrete and Mild Steel Bars : $\rho_{min} = 0.0035$

Other Concrete and Steel Reinforcement : $\rho_{min} = 0.06\, F_c / F_y$

Maximum Reinforcement

M15 Concrete and Mild Steel Bars : $\rho_{max} = \rho_c + 0.011$

Other Concrete and Mild Steel Bars : $\rho_{max} = \rho_c + 0.19\, F_c / F_y$

For Concrete Reinforced with Other Bars :

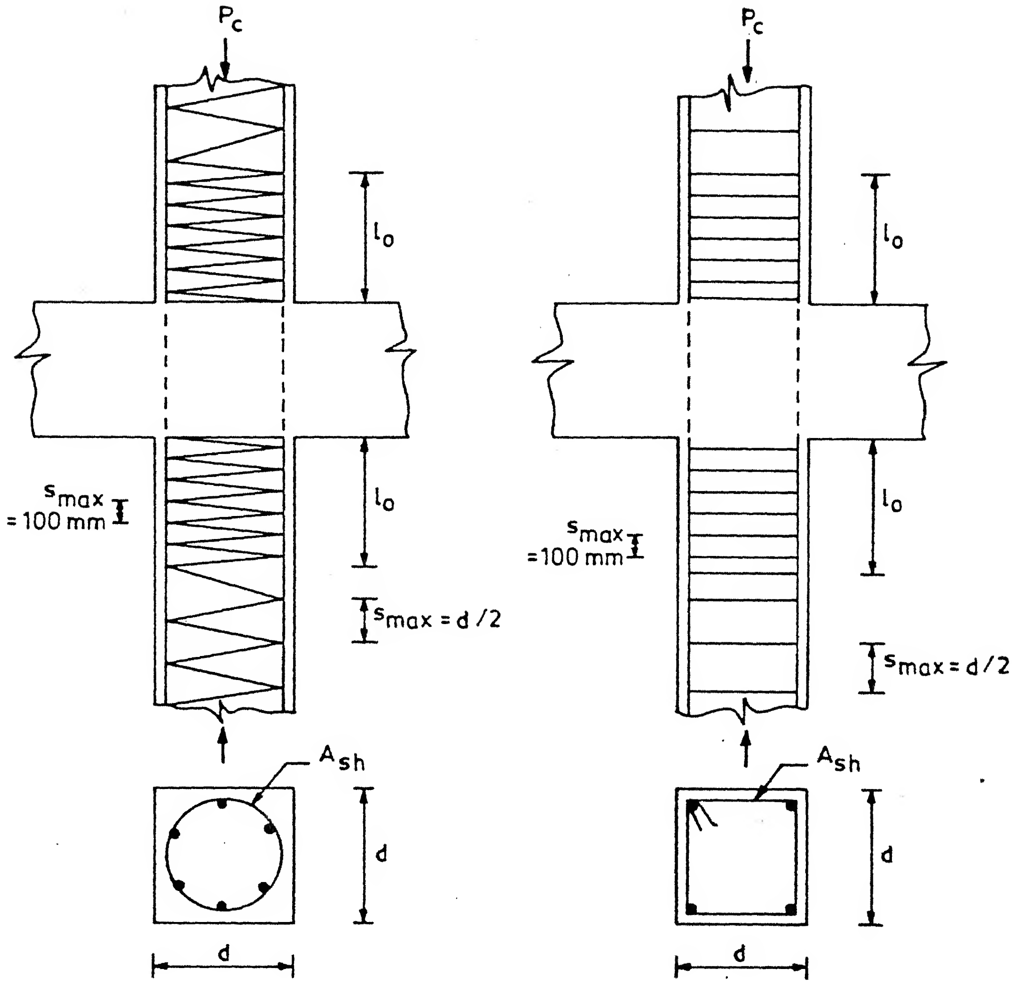
$$\rho_{max} = \rho_c + 0.15\, F_c / F_y$$

Web Reinforcement

Max. Spacing of Stirrups d/4 in a Length of 2d Near Each End of the Beam and d/2 in the Remaining Length.

*Splice to Be Contained Within At Least 2 Closed Stirrups, and Not to Be Provided at Sections of Max. Tension.

FIGURE 2. SPECIAL DUCTILITY DETAILS FOR FLEXURAL MEMBERS.



$$A_{sh} \geq 0.08\, s\, D_k \cdot \frac{F_c}{F_y} \cdot \left(\frac{A}{A_k} - 1\right)$$

(a) Spiral Confinement Reinforcement.

$$A_{sh} \geq 0.16\, s h \cdot \frac{F_c}{F_y} \cdot \left(\frac{A}{A_k} - 1\right)$$

(b) Rectangular Hoop Confinement Reinforcement.

$$l_o \geqslant \begin{cases} \text{1/6 Height of Column} \\ \text{Larger Lateral Dimension} \\ \text{450 mm} \end{cases}$$

FIGURE 3. SPECIAL DUCTILITY DETAILS FOR COLUMNS.

In the following paras, the codal provisions have been discussed at length. One of the publications of American Concrete Institute, ACI 318-83 (Ref.6) is a building code giving requirements for reinforced concrete, similar to IS:456. Its Appendix:A describes provisions for seismic design, as in Clause (7) of IS:4326. SEAOC Code (Ref.15), ANSI - 1982 (Ref.9) and UBC-1985 (Ref.8) recommend minimum design seismic loads for buildings and other structures, which include values for horizontal force factor, K. This factor has the same meaning as performance factor of IS:1893. IS code provisions have been compared with provisions followed in the U.S., where research in earthquake resistant design is in much more advanced stage than in our country. Also listed are some suggestions for inclusions.

4.1 PERFORMANCE FACTOR :

IS:4326 requires that the ductility requirements specified therein " shall be adopted in all cases where the design seismic coefficient α_h is 0.05 or more." Moreover, Clause (1.2) of the same code states, "The provisions of this standard are applicable for building construction in seismic zones III to V. No special provisions are necessary for building constructions in seismic zones I and II." However, IS:1893 - 1984 does not allow this distinction and even in seismic zones I and II increases seismic forces by 60% for non - ductile R.C. frames. Thus, there is an undue penalty which adds to the cost of construction even in areas which are seismically inactive. It is not reasonable to expect buildings in zones I and II to conform to ductile detailing at the same level as in zones IV and V. Thus it is surely a rather steep increase (of 60%) in design seismic forces for structures in low seismic zones. The code must be modified to remove this anamoly.

Even in seismically more active zones (III to V), one must carry out a thorough study on cost implications of the new provisions. For instance, a designer should have some idea of the order of magnitude of costs if he designs say a 4 - storey R.C. frame office/residential building in zone III as (i) ductile frame with K = 1.0; or as (ii) non - ductile frame with K=1.6. At present he does not have any such guidelines. Moreover the author's personal experience is that large number of designers in the country still do not design structures as ductile. Interaction with some designers in private as well as public sector indicates that even two years after the release of IS:1893 - 1984 (it was released in later part of 1986), they continue to use IS:1893 - 1975, thereby avoiding K = 1.6 for non - ductile frames. This tendency has been encouraged because of steep rise in the cost of the same building in the same zone if new provisions are used. There are bureaucratic pressures on designers to reduce cost of construction and he finds it difficult to justify this steep rise instead.

Another difficulty with inclusion of performance factor, K, is that as yet no provisions for ductile detailing of R.C. shear walls or steel bracing members are available in Indian codes, while IS:1893 - 1984 specifies K for such structures. Also, most multistorey construction in the country consists of R.C. frames with brick in-fill panels wherein the brick in-fills are assumed to be non-structural. Also, there is not enough experimental data available for brick in-fill and frame interaction and hence no guidelines are available on the lateral load response of such in-fills. However, the code seems to allow for incoporation of

the structural contribution of such in-fills. This may lead to misinterpretation and improper use by the designer.

The provision of performance factor for buildings and not for other structures has created a very obvious anamoly. In the absence of K for other structures (e.g., water tanks, chimneys, bridges, etc.), it amounts to taking K = 1.0 for such structures which is the same as for a building with ductile frames. However, such structures will not exhibit the same ductility as a ductile building, especially when there is no requirement for ductility detailing in other structures. In fact American codes specify a much higher value of K for other structures because "most of these other structures do not have the multiplicity of structural and non-structural resisting elements characteristics of most buildings; do not have significant natural damping; do not have elements which could be permitted to yield or even fail without jeopardizing the safety of the structure" (Ref.15). For instance, American codes use K = 2.0 for structures other than buildings as compared to K = 0.67 for buildings with a ductile frame, i.e., <u>three times</u> that for ductile frames. On the other hand, as per IS:1893, other structures have the same value of K as a ductile - frame building. This anamoly needs to be removed.

4.2 DUCTILITY DETAILING CRITERIA :

IS:4326 requires ductility detailing if α_h is 0.05 or more. This leads to difficulty in application. This is because, in the same city, depending on β and I, the value of α_h will vary. It is difficult to practice different detailing requirements in the same city by builders and engineers. Instead, if this requirement could be based on seismic zone alone, in due course a detailing culture will evolve in each

geographical area.

The design provisions contained in the main body of the ACI building code provide some ductility which is sufficient for structures subjected to only minor earthquakes that may occur frequently. For structures that may be subjected to earthquakes of moderate intensities, some additional confinement, anchorage and shear reinforcement details are required. For structures that may be subjected to strong intensity earthquakes, appreciable inelastic deformations can be expected so that substantial ductility is required. Special provisions in Appendix:A are intended to provide the additional ductility (Ref.13). Hence depending on the seismic risk of the zone, three levels of ductility are adopted by ACI, for any reinforced concrete construction. The main body of ACI covers ductility requirements for zones of low seismic risk (equivalent to zones I and II in India) and ACI Appendix:A contains different provisions for zones of moderate seismic risk (equivalent to our zone III) and zones of high seismic risk (equivalent to our zones IV and V).

If we see the provisions of IS codes from this angle of view, prior to 1984, the IS:1893 in effect had performance factor, K, equal to unity and the detailing had to be adopted depending on α_h . With the fourth edition of IS:1893 in 1984, performance factor, K, has been introduced, which gave the designer two broad options. One is to provide ordinary concrete frames but with enhanced lateral loads and second to provide ductile concrete frames with lower value of lateral loads. The ductility detailing is to be adopted only for the latter case. This option could allow a designer to provide ordinary concrete frames even in zone V, which is quite contrary to the essence of

IS 4326:1976. Again, structures designed to be built in zones IV and V will be extremely uneconomical if ordinary concrete frames are used with enhanced lateral loads. It would be very appropriate to include the three levels of ductility requirements for Indian conditions also. The balanced percentage of steel reinforcement as calculated by ACI code for flexural members is much more than that calculated by IS:456, for same grade of steel and concrete. That means ACI defines the balanced condition for the flexural sections with more steel ratio than that defined in IS:456 and yet, its provisions of main body are said to provide sufficient ductility in zones of low seismic risk. Generally for economy, the designs of members are aimed to be done with steel ratios close to balanced steel ratios. Hence, members designed with IS:456 provisions are likely to have less steel ratios and better ductility as compared to sections designed by ACI code. Hence, for zones of low seismic risk (zones I and II) it can be safely said that provisions of IS:456 will provide sufficient ductility. The other two levels of ductility for zones of moderate and high seismic risk are to be included in our codes.

4.3 MATERIALS :

There are no provisions in the code for maximum grade of steel and minimum grade of concrete. Minimum grade of concrete of M20 may be considered to be included in zones IV and V. IS:456 mentions the use of high yield strength deformed bars of grade Fe500, but these are not commonly used in India. However a maximum grade of steel of Fe415 may be considered to be specified. Limitation on maximum variation of actual yield

strength of longitudinal steel provided in the structure, to that specified by the designer may also be recommended. If the variation is large, shear in the flexural members at the time of formation of plastic hinges will be very high and may cause a brittle shear failure. These limits are necessary in view of the unfavourable effects, the decrease in concrete strength and increase in yield strength of steel have on the sectional ductility of members in which they are used (e.g., Refs. 13, 14).

4.4 STRONG COLUMN - WEAK GIRDER DESIGN:

As mentioned earlier, it is important in earthquake resistant design that hinges must form in beams and not in columns. Hinge formation in columns leads to early collapse of the structure. To reduce likelihood of yielding in columns, ACI 318-83 requires the flexural strength of columns (in regions of high seismic risk) to satisfy

$$\Sigma M_c \geq (6/5) \Sigma M_g \qquad \ldots (4)$$

where, ΣM_c = sum of moments, at the centre of the joint, corresponding to the design flexural strength of the columns framing into that joint. Column flexural strength to be calculated for the direction of the lateral forces considered, resulting in the lowest flexural strength.

ΣM_g = sum of moments, at the centre of the joint, corresponding to the design flexural strengths of the girders framing into that joint.

However, Indian codes do not have any such provision. It is highly desirable to incorporate such a provision in IS:4326 for

columns of buildings in zones IV and V.

4.5 FLEXURAL MEMBERS:

(i) Definition :

Members with average axial stress P/A under earthquake condition less than 0.1 F_c are to be treated as flexural members. ACI code also has a similar requirement. However, ACI code also specifies restrictions on the sectional dimensions which are not specified in Indian code. But none of the codes have minimum specified depth for a flexural member. A minimum depth of beam is desirable to be specified because, beams with depth (d) about 300 mm (common in residential buildings) will have clear spacing between stirrups less than 75 mm at the potential locations of plastic hinges, to satisfy the maximum spacing (d/4) requirement. This could cause constructional difficulties in placing and compacting the concrete.

(ii) Longitudinal Reinforcement :

The minimum longitudinal reinforcement recommendations given in IS:4326 (e.g.,Fig.2) are based on minimum ductility provisions as given in Ref.12. However, ACI code recommends higher values for minimum longitudinal reinforcement for the same grade of concrete and steel and ACI provisions seem more reasonable. The minimum steel ratios in ACI code were found by equating the cracking moment of the section of the plain concrete to the moment capacity of the reinforced concrete section and solving for steel ratio (Ref.14). As a beam is loaded, initially the gross concrete section resists the moment while steel does not get stressed much. However, beyond cracking moment, the tension concrete starts cracking. Immediately after the concrete

cracks, section capacity must not be less than what it was before cracking started otherwise the beam will fail suddenly in a brittle manner. Hence, it is recommended that the IS provisions for minimum longitudinal reinforcement be revised upwards following the same criteria as in ACI code.

The maximum tensile steel ratios recommended in Clause (7.2.2) are from Ref.12 and are dependent on grade of steel and concrete. ACI recommends a fixed value of 0.025, which is based on criterion of congestion of steel reinforcement.

(iii) Moment capacity in the member :

ACI recommends positive strength at joint face to be greater than one - half of the negative moment strength provided at that face of the joint. Throughout the member, the moment strength is to be greater than one fourth the maximum strength provided at the face of either joint. This provision is lacking in IS:4326. Since the earthquake loads are reversible in nature, inclusion of this clause may be considered.

(iv) Lap Splices :

Considering that the failure in bond between steel and concrete is brittle, and lap splices are not reliable under conditions of cyclic loading into the inelastic range, ACI recommends stringent provisions for locating lap splicings and enclosing them in transverse steel. IS code provisions are incomplete in this aspect.

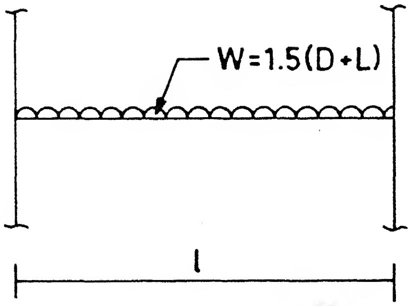
(v) Design Shear Force :

Avoiding shear failure is one of the most important requirements of earthquake resistant design. This is achieved in IS:4326 through Clause (7.2.5) which reads, " The web reinforcement in the form of vertical stirrups shall be provided

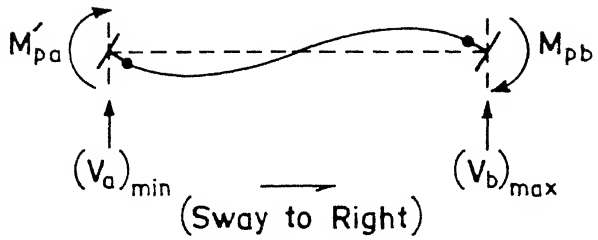
vertical loads acting on the beam plus those which can be produced by the plastic moment capacities at the ends of the beam. The spacings of the stirrups shall not exceed d/4 in a length equal to 2d near each end of the beam and d/2 in the remaining length ."

There appears to be a lack of understanding of this clause among some design engineers. It was noticed that some of them did not quite follow the first sentence of this clause and assumed that the second sentence ensures compliance of requirement specified in the first sentence. Handbook SP:22 (Ref.5) has explained some of these requirements. But the expressions given in SP:22 for minimum and maximum design shear forces are incorrectly printed. The correct version is given in Fig.5 of this Part-I. In conventional design, a beam is designed for moments and shears obtained from analysis for given loads. However, this clause requires shear design from a different viewpoint and is meant to ensure that the beam does not fail in shear before formation of plastic hinges. This is to avoid brittle shear failure. The code should bring this out much more clearly and should preferably separate these two sentences into separate clauses.

This clause for shear reinforcement design has two deficiencies.Firstly it does not account for the effect of strain hardening in longitudinal reinforcement. Research worldwide has established that the main steel in fact goes into its non-linear range during severe earthquakes and shear forces may increase to a large extent. Secondly, it is not clear how to calculate "plastic moment capacities at the ends of the beam" as specified in IS:4326. For instance, what stress must be taken in steel and

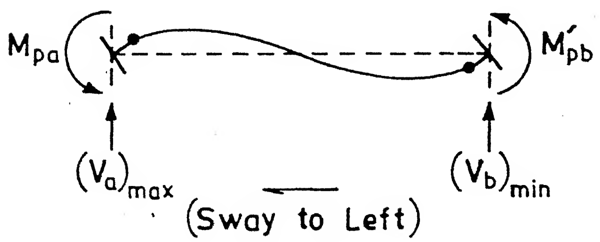


$$V_a^{D+L} = V_b^{D+L} = \frac{W}{2}$$



$$(V_a)_{min} = V_a^{D+L} - \frac{M'_{pa} + M_{pb}}{l}$$

$$(V_b)_{max} = V_b^{D+L} + \frac{M'_{pa} + M_{pb}}{l}$$



$$(V_a)_{max} = V_a^{D+L} + \frac{M_{pa} + M'_{pb}}{l}$$

$$(V_b)_{min} = V_b^{D+L} - \frac{M_{pa} + M'_{pb}}{l}$$

FIGURE 5. LOADING CONDITION FOR DESIGN OF SHEAR REINFORCEMENT IN FLEXURAL MEMBERS. (UNIFORMLY DISTRIBUTED LOADING)

concrete at this condition. ACI code follows the ultimate strength philosophy of design, with appropriate strength reduction factors. So its simple reference to the plastic moment capacity as that corresponding to probable strength using the properties of the members at the joint faces without strength reduction factors and assuming that the stress in the tensile reinforcement is equal to at least 1.25 fy, is adequate. IS code may stipulate plastic moment capacity as that corresponding to γ_m, partial safety factor for material strength, equal to unity and stress in the tensile reinforcement equal to at least 1.25 fy.

Deterioration of shear strength of concrete owing to alternate opening and closing of cracks during repeated reversal of deformations in non-linear range (effect of 'shear sliding') may also be taken into account by specifying zero shear strength capacity of concrete for large shear forces associated with the formation of plastic hinges at the ends as in ACI.

Handbook SP:22 on the other hand must contain clear figures (such as Fig.5) of deformed shape of flexural members and the expressions for the design shear forces. It should contain actual procedure as outlined in Annexure:A and tables corresponding to plastic moment capacities of sections for different grades of steel and concrete similar to those given in SP:16 (Ref.11) to enable the designer to use them directly without wasting time on lengthy calculations.

Studies have revealed (Ref.Part II) that, for moderately reinforced sections with slab live loads 3 to 5 kN per square metre, 8 mm tor steel provided at maximum spacing of d/4, almost always governs the design of transverse steel, i.e., the shear caused by plastic hinges does not govern in most cases, if the

shear capacity of concrete in the section is taken into account.

The possibility of correlating the curves and tables already available in SP:16 with the plastic moment capacities of the sections is to be thoroughly explored to save designer's time.

(vi) Minimum Diameter of Transverse Steel :

It is also suggested that in zones IV and V, minimum bar diameter of 8 mm be specified for transverse reinforcement. This is because percentage reduction of the gross sectional area of 6 mm bars, due to rusting of the ribs and surface, upto placing of concrete is much more than that of bars of higher diameter. Hoops are required not only to provide shear strength, so that full flexural capacity of the member can be developed, but also to help ensure adequate rotation capacity at plastic hinging region by confining concrete in the compression zone and by providing lateral support to the compression steel. Thus, the improved ductile behaviour of the member due to higher diameter stirrups would be disproportionately large as compared to the negligible increase in the quantity of steel. It may be mentioned here that ACI provides for a minimum #3 bar (i.e.,10mm dia) to be used as transverse steel even in ordinary concrete frames.

(vii) Diagonal Shear Reinforcement :

Clause (7.1.4) of IS:4326 contains provisions for limiting the value of maximum shear carrying capacity of diagonal bars to 50% of the design shear. This is a provision similar to that given in British code CP:110 (Ref.10). However, Britain is not seismically active. ACI does not allow even 50% contribution of diagonal bars. Keeping in view the possible shear reversal during

an earthquake, disallowing the use of diagonal bars may be considered for zones IV and V.

4.6 COLUMNS SUBJECTED TO AXIAL LOADS AND BENDING :

(i) Definition :

Clause (7.3.1) defines "columns subjected to axial load and bending" if the member is subjected to average stress P/A greater than 0.1 F_c. IS code may consider reclassifying this group as "members subjected to axial load and bending", as members satisfying these conditions need not necessarily be columns. ACI code gives additional requirements on section sizes of members subjected to axial load and bending. Of particular importance is the one restricting shortest dimension of the cross-section measured on a straight line passing through the geometric centroid to be not less than 300 mm.

(ii) Confinement Reinforcement :

Clause (7.3.2) of IS:4326 specifies the minimum amount of confinement reinforcement for spiral and rectangular closed hoops. These expressions are comparable to the ACI recommendations. (The difference is because ACI refers to cylinder strength, while IS code refers to cube strength.) For circular hoops or spirals used for confinement of concrete, IS code requires,

$$A_{sh} = 0.08\ s\ D_k \frac{F_c}{F_y} \left[\frac{A}{A_k} - 1.0 \right] \qquad \ldots(5)$$

On the other hand Clause (38.4.1) of IS:456 requires for all constructions the ratio of volume of helical reinforcement to the

volume of the core (ρ_s) shall not be less than 0.36 $\left[\frac{A_g}{A_c} - 1.0 \right] \frac{f_{ck}}{f_y}$

since $$\rho_s = \frac{\Pi \; D_k \; A_{sh}}{\frac{\Pi}{4} \; D_k^2 \; s}$$

therefore $$A_{sh} = 0.09 \; s \; D_k \frac{f_{ck}}{f_y} \left[\frac{A_g}{A_c} - 1.0 \right] \qquad \ldots(6)$$

Thus, IS:4326 requirement is less than that of IS:456 which is inconsistent. Hence, expression in IS:4326 can be either removed or its provision may be revised upwards to match that in IS:456. The requirement for confinement by the rectangular hoops is derived from this requirement for spiral reinforcement. Hence, for rectangular hoop reinforcement, corresponding value of cross sectional area in IS:4326 may be changed from

$$A_{sh} = 0.16 \; s \; h \; \frac{f_{ck}}{f_y} \left[\frac{A_g}{A_c} - 1.0 \right] \qquad \ldots(7)$$

to, $$A_{sh} = 0.18 \; s \; h \; \frac{f_{ck}}{f_y} \left[\frac{A_g}{A_c} - 1.0 \right] \qquad \ldots(8)$$

A minimum volumetric ratio of spiral or circular hoop reinforcement, $\rho_s = 0.12 \frac{f'_c}{f_{yh}}$ is recommended by ACI to specify a lower bound which governs for larger columns with gross cross sectional area, Ag, less than approximately 1.25 times the core area, A_c (Ref.7). Here f'_c is cylinder strength which may be related to cube strength as $f_{ck} = 0.85 \; f'_c$. IS code may incorporate this requirement as $\rho_s = 0.14 \frac{f_{ck}}{f_y}$. For rectangular hoops, ACI recommends minimum total cross - sectional area

$A_{sh} = 0.12\ s\ h\ \frac{f'_c}{f_{yh}}$ which may be adopted in IS:4326 as

$$A_{sh} = 0.14\ s\ h\ \frac{f_{ck}}{f_y}$$

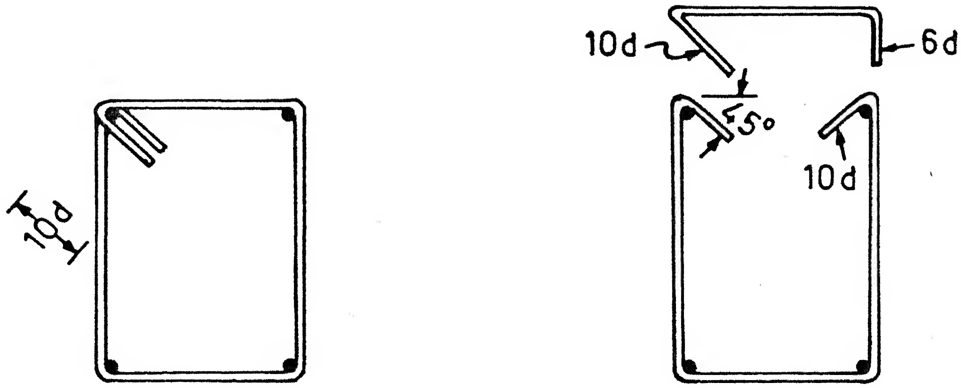
(iii) Shear Reinforcement :

Clause (7.3.4) of IS:4326 requires provision of shear reinforcement to resist shear resulting from the lateral and vertical loads at ultimate load conditions of the frame and specifies a maximum spacing of d/2 throughout the member. This implies checking for shear reinforcement in addition to minimum confinement steel. However, it does not clearly mention what is meant by "ultimate load conditions". ACI code requires the design shear force, V_c, to be determined from consideration of the forces on the member, with the nominal moment strengths calculated for the factored axial compressive forces, resulting in the largest moment, acting at the face of the joints. It specifies the nominal moment strength as the limiting moment of the section with strength reduction factors equal to unity.

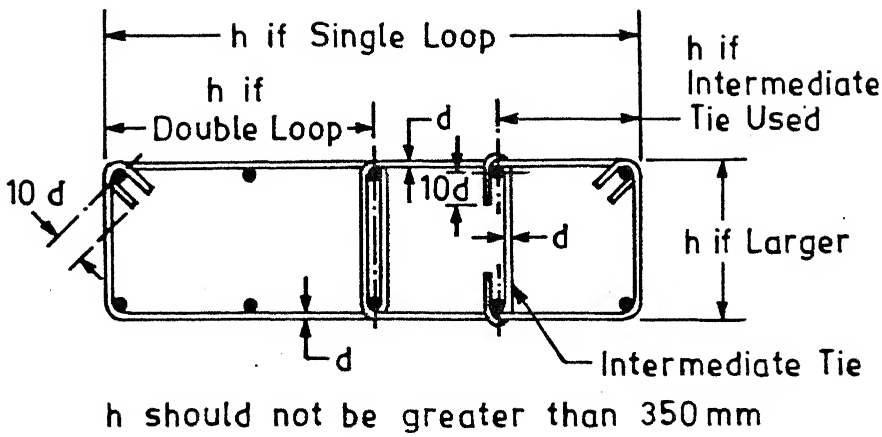
If such a requirement is to be specified in IS code, the procedure to arrive at nominal moment capacity must be included. It may say that the nominal moment capacity is to be arrived at by setting γ_m, partial safety factor for materials equal to one.

(iv) Development Length of Hoops :

In seismic design, the development length or anchorage of closed stirrups or hoops is usually more than that for ordinary concrete construction. The members rely on full development of yield strength in transverse steel for confinement of concrete and rotation capacity of the members. Providing closed hoops in beam - column joints also causes



(a) Details of Hoops (From Ref.- 9)



(b) Details of Hoops for Columns and Dimension h in Rectangular Hoop (Modified from Ref.- 2)

FIGURE 4. DETAILS FOR TRANSVERSE REINFORCEMENT.

constructional difficulties. ACI allows hoops in two pieces (Fig.4a) with a stirrup having 135° bends and ten diameter extensions, anchored in the confined core and a crosstie to make a closed hoop. Such hoops are uncommon in Indian construction industry, but these are highly essential to enable the placement of transverse steel in joints. Further IS:4326 does not have provision for extra extension beyond the bends in stirrups for ductile frame members. In Clause (25.2.2.4b) IS:456 requires continuation of stirrup or transverse steel for 8 diameters for 90° bend, 6 diameters for 135° bend or 4 diameters for 180° bend. On the other hand IS:4326 seems to require extension of 10 diameters for closed hoops and intermediate ties in column sections as implied from its Fig.5, but it does not explicitly spell out this requirement. There is altogether no such provision for hoops in flexural members. In the absence of an explicit provision in this regard, the industry would continue using detailing as per IS:456. Hence this provision needs to be explicitly stipulated in the form of a clause.

(v) Minimum Diameter of Transverse Steel :

For reasons listed earlier, a minimum of 8 mm diameter may be specified for hoops required for confinement. Transverse steel results in improvement of both strength and ductility alongwith prevention of buckling of longitudinal steel, keeps the core concrete confined and prevents shear failure from propagating through the section. Higher transverse reinforcement also helps in fully developing required ultimate curvature. Fig. 5 of IS:4326 shows the details of rectangular hoops to be provided, but it has a misprint showing the diameter of one of the hoops to be equal to 10d instead of d. The same has been

rectified and shown in Fig. 4b.

(vi) Columns Supporting Reactions from Discontinuous Stiff Members :

ACI has additional provisions for columns supporting reactions from discontinuous stiff members, which are not present in IS code. Such provisions are highly desirable for zones IV and V in India.

4.7 BEAM-COLUMN CONNECTIONS :

Clause (7.1.3) in IS:4326 says that the beam column connections shall preferably be made monolithic. However field and laboratory experience which has led to ductile detailing requirements has been predominantly with monolithic reinforced concrete building structures (Ref.7). So, use of 'preferably' in this clause is not warranted.

IS code limits the amount of transverse reinforcement in beam-column connections to that required at the ends of the columns. ACI code requires that the shear capacity be checked as per the actual forces caused by taking into account the column shears and the shears developed from yield forces in beam reinforcement after reaching strain hardening state. It clearly quantifies shear strength in joints and gives requirements of lapping for beam reinforcement. IS code requirement of transverse reinforcement in joints as presently specified needs to be thoroughly reviewed and revised.

4.8 GENERAL :

ACI code also contains provisions for structural walls,

diaphragms and trusses and provisions for frame members not proportioned to resist forces induced by earthquake motions which IS code is lacking.

Handbook SP:22, can play a very vital role in spreading the letter and spirit of IS:4326 in engineering profession in the country. It should encourage the designer to opt for ductile frames in zones IV and V and to design structures which are uniform in strength and ductility throughout rather than with individual and isolated over-strong members. In ordinary design for static loads, the presence of overstrong members does not decrease the strength of the structure. In seismic design, however, when structure relies on energy dissipation by ductile plastic hinges to survive earthquakes, the presence of overstrong members leads to collapse because of the very high inelastic deformations enforced elsewhere. The handbook should illustrate use of IS:4326 with examples and neat detailing methods. It should contain design tables and procedures for determination of plastic moment capacity and the ultimate moment capacity of sections.

It should also guide the designer to provide minimal reinforcement, which avoids steel congestion in joints and members, and also provides better ductility. The designer should also be encouraged to lavishly use transverse reinforcements in the frame members because, extra ties and hoops are inexpensive due to their low weight and minimal fabrication costs but their use can substantially better the performance both in ductility and confinement.

5 CONCLUSIONS :

A structure is expected to go into its inelastic range of response during a severe earthquake and so, needs large ductility for efficient dissipation of energy. Ductility provisions of IS codes have been reviewed and compared with those in ACI code. The following major suggestions have been made.

(i) Rationalization of performance factor, K, specified in IS:1893 - 1984.

(ii) Introduction of ductility detailing depending on zones rather than α_h .

(iii) Introduction of three levels of ductility requirements, i.e., ductility detailing for zones of low , moderate and high seismic risks.

(iv) Inclusion of limits on strength of concrete and steel used in structures.

(v) Introduction of strong column-weak girder concept in the design of structures.

(vi) Revision of minimum longitudinal reinforcement criteria, provisions for design of transverse reinforcement in flexural members, etc.

(vii) Inclusion of guidelines for arriving at plastic moment capacity of sections.

(viii) Modification of amount of confinement steel in columns.

(ix) Inclusion of minimum diameter for transverse reinforcement in columns and beams.

(x) Inclusion of provisions for design of beam - column joints

specifying the design shear force on joints and the shear capacity of joints.

(xi) Inclusion of provisions for structural walls, diaphragms, trusses, etc.

(xii) Revision of SP:22 for clear explanation on seismic code provisions alongwith numerical illustrations and design tables for plastic moment capacity.

APPPENDIX: A

PROCEDURE FOR CALCULATION OF PLASTIC MOMENT CAPACITY FOR R.C. BEAMS

Steps for calculation of plastic moment capacity of reinforced concrete section have been given below. From any standard book (Refs. 16,17) ón design of reinforced concrete structures, following relations in accordance with IS:456 can be derived. The effect of strain hardening in tension steel has been incorporated in these relations. Taking $\varepsilon_c = 0.0035$,

$$\varepsilon_y = 0.002 + \frac{f_s}{\gamma_m . E}, \quad f_s = 1.25\ f_y, \quad \gamma_m = 1.0, \quad E = 2.0 \times 10^5 \text{ MPa},$$

for balanced failure,

$$x_{umax} = \frac{35d}{55 + 0.0625\ f_y} \qquad \ldots(1)$$

$$z_{max} = (d - 0.416\ x_{umax}) \qquad \ldots(2)$$

$$\frac{M_{ulim}}{bd^2} = 0.54\ f_{ck} \left[\frac{x_{umax}}{d}\right] . \left[\frac{z_{max}}{d}\right] \qquad \ldots(3)$$

$$p_{lim} = 43.2 \left[\frac{x_{umax}}{d}\right] . \left[\frac{f_{ck}}{f_y}\right] \qquad \ldots(4)$$

For an under reinforced section:

$$x_u = 2.315 . \left[\frac{f_y}{f_{ck}}\right] . \left[\frac{p_t}{100.0}\right] . d \qquad \ldots(5)$$

$$\frac{z}{d} = \left[1.0 - 0.416 \frac{x_u}{d}\right] = \left[1.0 - 0.963 \frac{f_y}{f_{ck}} . \frac{p_t}{100.0}\right] \qquad \ldots(6)$$

$$\frac{M_p}{bd^2} = 1.25\ f_y \cdot \frac{z}{d} \cdot \frac{p_t}{100.0} \qquad \ldots(7)$$

where :

ε_c - Useful limit of strain in concrete.

ε_y - Useful limit of strain in steel.

f_s - Stress in steel.

E - Young's Modulus of steel.

γ_m - Partial safety factor for material strength.

x_u - Depth of neutral axis.

x_{umax} - Limiting value of x_u.

d - Effective depth of section.

d' - Cover to compression steel.

f_y - Characteristic strength of reinforcement.

f_{sc} - Stress in compresion steel.

f_{ck} - Characteristic strength of concrete.

b - Width of the compression face.

$M_{u,lim}$ - Limiting moment of resistance of a section without Compression steel.

z - Lever arm.

z_{lim} - Limiting value of lever arm, z.

p_c - Percentage area of compression reinforcement.

p_t - Percentage area of tension reinforcement.

p_{lim} - Limiting value of tension reinforcement in singly reinforced sections.

ε_{sc} - Strain at the level of compression steel.

M_p - Plastic moment capacity of the section.

Using the above equations and the steps given in Fig.(A1) design tables and charts can be prepared as in SP:16 and included in SP:22. A brief parametric study, not described here, has revealed that the plastic moment capacity of an under reinforced section is about 44% to 50% higher than its design moment capacity. Corresponding increase in capacity asper ACI is about 40%. In addition, ACI also reduces the allowable shear stresses by 40% by about specifying strength reduction factor of 0.60 for ductile frames as against 0.85 for ordinary concrete frame members.

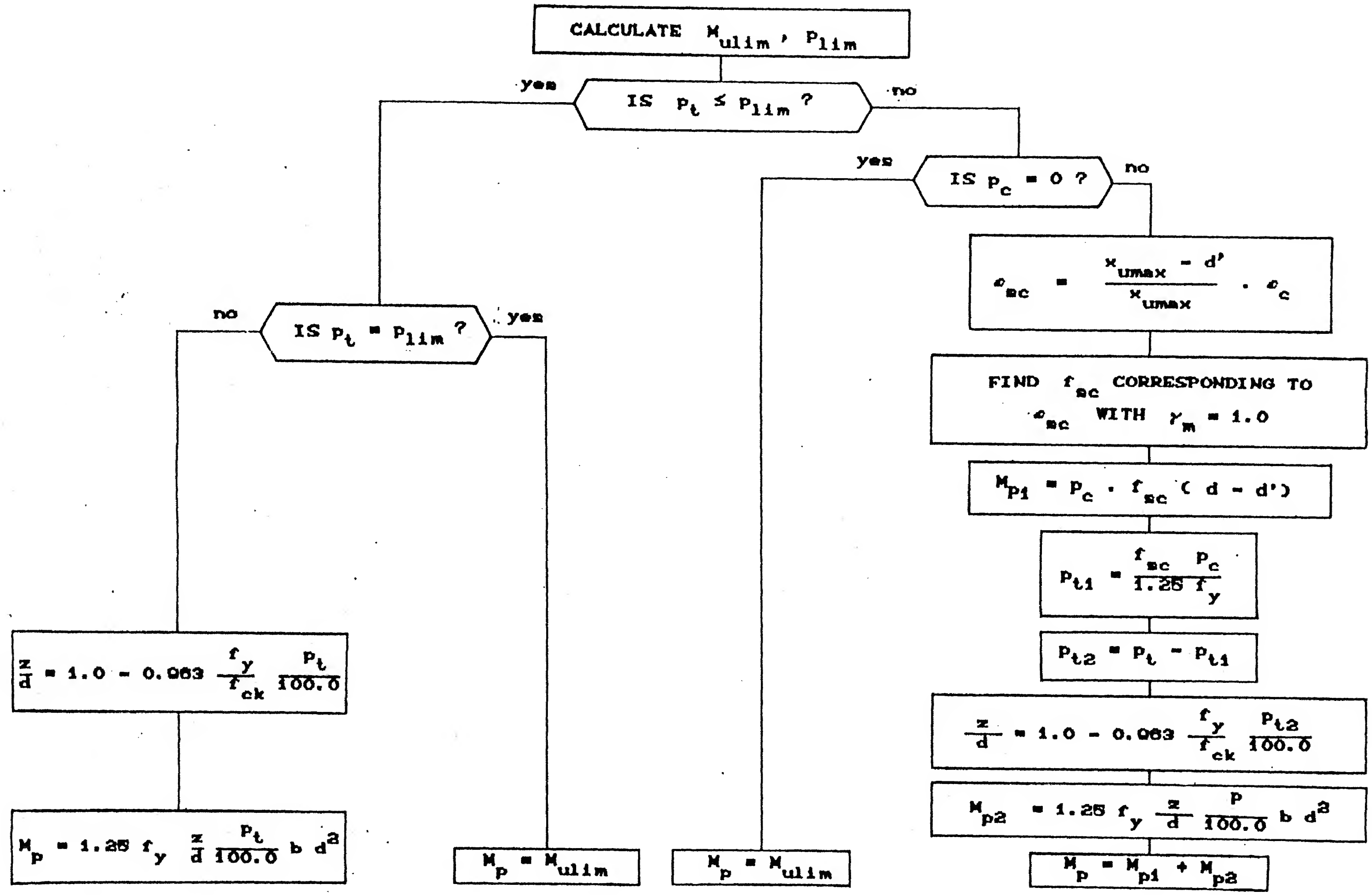


FIGURE A1. STEPS FOR CALCULATION OF PLASTIC MOMENT CAPACITY OF R.C. BEAMS.

APPENDIX B : REFERENCES

1. IS: 1893 - 1984, Indian Standard Criteria for Earthquake Resistant Design of Structures, Bureau of Indian Standards New Delhi.

2. IS: 4326 - 1976, Indian Standard Code of Practice for Earthquake Resistant Design and Construction of Buildings, Bureau of Indian Standards, New Delhi.

3. IS: 456 - 1978, Indian Standard Code of Practice for Plain and Reinforced Concrete, Bureau of Indian Standards , New Delhi.

4. IS:1893 - 1975, Indian Standard Criteria for Earthquake Resistant Design of Structures, Bureau of Indian Standards, New Delhi.

5. SP : 22 (S&T) - 1982, Explanatory Handbook on Codes for Earthquake Engineering, IS: 1893 - 1975 & IS: 4326 - 1976, Bureau of Indian Standards, New Delhi.

6. ACI 318 - 83, Building Code Requirements for Reinforced Concrete, American Concrete Institute,Detroit, Michigan 48219, USA.

7. ACI 318 - 83 R, Commentry on Building Code Requirements for Reinforced Concrete, American Concrete Institute, Detroit, Michigan 48219, USA.

8. Uniform Building Code, 1985 edition, International Conference of Building Officials, 5360 South Workman Mill Road, Whittier, CA 90601, USA.

9. ANSI A58.1 - 1982, Minimum Design Loads for Buildings and other Structures, American National Standards Institute, 1430, Broadway, New York, NY 10018.

10. CP110 : Part I : 1972, Code of Practice for the Structural Use of Concrete, British Standards Institution, London.

11. SP:16 (S & T) - 1980, Design Aids for Reinforced Concrete to IS : 456 - 1978, Bureau of Indian Standards, New Delhi.

12. Blume,J.A., N.M.Newmark and L.H.Corning, 1961, Design of Multistorey Reinforced Concrete Buildings for Earthquake Motions, Portland Cement Association, Skokie, Illinois, USA.

13. Derecho, A.T., M. Fintel and S.K. Ghosh, 1985, Earthquake Resistant Structures, in Handbook of Concrete Engineering, Mark Fintel (Ed.), Van Nostrand Reinhold, New York.

14. Park,R and T.Paulay, 1975, Reinforced Concrete Structures, John Wiley & Sons, New York.

15. SEAOC, 1980, Recommended Lateral Force Requirements and Commentry, Seismology Committee, Structural Engineers Association of California, 171 Second Street, San Francisco, California 94105.

16. Jain, A.K., 1984, Reinforced Concrete Limit State Design, second edition, Nem Chand & Bros., Roorkee.

17. Dayaratnam, P., 1984, Design of Reinforced Concrete Structures, second edition, Oxford & IBM Publishing Co., New Delhi.

18. Esteva, L., 1980, 'Design : General', in Design of Earthquake Resistant Structures, E. Rosenblueth (Ed.), Pentech Press, London.

19. Nmai, C.K. and D. Darwin, 1986, 'Lightly Reinforced Beams Under Cyclic Load,' Journal of ACI, Vol. 83, Sep.- Oct., pp. 777 - 784.

20. Kent,D.C., and R. Park, 1971,'Flexural Members with Confined Concrete ', Journal of Structural Division, ASCE , ST7, July, pp.1969 - 1990.

21. Brown, H.R, and J.O.Jirsa, 1971, 'Reinforced Concrete Beams Under Load Reversals', Journal of ACI, Vol.68, May - June, pp.380 - 390.

22. Robbat, B.G., J.I. Daniel, T.L. Weinmann, and N.W. Hanson, 1986, 'Seismic Behaviour of Light Weight and Normal Weight Concrete Columns', Journal of ACI, Vol.83, Jan.-Feb., pp.69-80.

23. Lawrence, S., I.Martin, R.Park, and L.Wyllie, 1980, 'Strong and Tough Concrete Columns for Seismic Forces', Journal of Structural Division, ASCE , ST8, Aug., pp.1717 - 1734.

24. Sheikh, S.A., and S.M.Uzumeri, 1980, 'Strength and Ductility of Tied Concrete Columns', Journal of Structural Division, ASCE, ST5, May, pp. 1079 - 1102.

25. Wight, J.K., and M.A. Sozen, 1975, 'Strength Décay of RC Columns Under Shear Reversals', Journal of Structural Division, ASCE, ST5, May, pp.1053 - 1065.

26. Park, R. and R.A. Sampson, 1972, 'Ductility of Reinforced Concrete Column Sections in Seismic Design', Journal of ACI, Vol 69, Sep.- Oct., pp.543 - 551.

27. Burdette, E.G., and H.K. Hilsdori, 1971, 'Behaviour of Laterally Reinforced Concrete Columns', Journal of Structural Division, ASCE, ST2, Feb., pp.587 - 602.

28. ACI - ASCE 352, 1982, 'Recommendations for Design of Beam - Column Joints in Monolothic Reinforced Concrete Structures', Journal of ACI, Vol.82, May - June, pp. 266 - 283.

29. Meinheit, D.F. and J.O.Jirsa, 1981, 'Shear Strength of R/C

Beam - Column Connections,'Journal of Structural Division, ASCE, ST11, Nov., pp.2227 - 2245.

30. Salvador, M., A.H. Nilson and F.O. Slate, 1984, 'Spirally Reinforced High Strength Concrete Columns', Journal of ACI, Vol.81, Sep.-Oct., pp. 431 - 442.

31. Popov., E.P., 1984, 'Bond and Anchorage of Reinforcing Bars Under Cyclic Loading', Journal of ACI, Vol.81, July - Aug., pp. 340 - 348.

32. Jain, A.K., 1980, 'Review of Seismic Code Provisions for Concrete Buildings', Indian Concrete Journal, Vol.54, No.11,Nov., pp. 294 - 300.

33. Arya, A.S., 1986, 'Code Provisions for Ductile Design of Rectangular Reinforced Concrete Columns', 8th Symposium on Earthquake Engineering, University of Roorkee, Roorkee, 2nd Volume, pp. 173 - 184.

34. Clough, R.W. and J. Penzien, 1982, 'Dynamics of Structures', McGraw - Hill Book Company, New York, pp. 602 - 603.

APPENDIX C : LIST OF NOTATIONS

Symbols have been defined where they first appear and they are summarised here. symbols of Appendix:A are given in the text itself.

A - Gross Concrete area of the column section.

A_k - Area of Concrete core $= \dfrac{\Pi\, D_k^2}{4}$

A_{sc} - Area of compression steel.

A_{sh} - Area of hoop reinforcement.

A_{st} - Area of tension steel.

d - Effective depth of the section

D_k - Diameter of core

F_c, f_{ck} - Characteristic strength of concrete (IS code)

f'_c - Specified cylinder strength of concrete (ACI)

f_y, F_y - Specified yield strength of steel.

h - Dimension of the stirrups

I - Importance Factor as per IS:1893

K - Performance factor as per IS:1893

M_{pa} - Hogging moment capacity at A

M'_{pa} - Sagging moment capacity at A

M_{pb} - Hogging moment capacity at B

M'_{pb} - Sagging moment capacity at B

s - Spacing of transverse steel

s_{max} - Maximum spacing of transverse steel

α_h - Design seismic coefficient

ρ - Reinforcement ratio

ρ_{min} - Minimum reinforcement ratio

ρ_{max} - Maximum reinforcement ratio

β - Coefficient depending on soil-foundation system (IS:1893)

PART II

DESIGN AND COST IMPLICATIONS OF DUCTILITY REQUIREMENTS OF IS CODES FOR R.C.FRAMED STRUCTURES

1. INTRODUCTION :

A reinforced concrete multistorey framed structure can have frames with two types of detailing, i.e., frames with member design conforming to special ductility detailing of IS:4326-1976 (Ref.3), hereafter referred to as Ductile Concrete Frame (D.C.F) and frames with members conforming to requirements of IS:456-1978 (Ref.4), hereafter referred to as Ordinary Concrete Frame (O.C.F). With fourth edition of IS:1893 (Ref.1) performance factor, K, has been roduced in assessing the lateral loads to be applied in earthquake resistant design of structures. Prior to this in IS:1893-1975 (Ref.2), K was in effect equal to unity for all structures. D.C.F was to be provided if the value of design seismic coefficient, α_h, is 0.05 or more. But now the code (Ref.1) allows designer a choice to either provide D.C.F with same lateral loads that existed before 1984 (i.e., K=1.0), or to provide an O.C.F with sixty percent enhancement of lateral loads as compared to what existed earlier (i.e., K=1.6). The choice of type of frames to be provided in a structure and its cost implication will vary widely with zones, soil-foundation system, utility, type of structural system of building, dead and live loads, etc.

It may be mentioned here that, if D.C.F is provided, lateral loads in calculating member forces are lesser and the quantity of transverse steel and sometimes compression steel is more. However, if O.C.F is provided, the lateral loads are higher, hence main reinforcement in structure increases. There is a need to clearly study and understand the relative

cost of structures if one were to use O.C.F or D.C.F . It is useful to know the relative increase in cost with the fourth edition of IS:1893. It is also very important to have an idea of premium involved in providing earthquake resistance of structures. These aspects have been investigated in the present work for three practical buildings having significant difference in design features.

2. COST OF EARTHQUAKE RESISTANT CONSTRUCTION :

Some government organizations such, as CPWD, recommend extra cost for resisting earthquake forces for preliminary cost estimates. However, as such no other systematic study seems to have been made for evaluating cost of earthquake resistant construction in Indian conditions. A committee of experts set up by the Structural Engineers Association of California (SEAOC) has evaluated such additional costs for U.S. conditions (Ref.5). Table 1 summarises the estimated increase in cost to provide earthquake resistance according to this committee. These are based on extensive work as well as expert judgement of the committee. The assumptions on which this study was based are listed below:

1. That the minimum non-earthquake resistant structure in various localities is nevertheless properly designed to resist vertical load and wind load forces as recommended by the Uniform Building Code (UBC) or other comparable model code, and the construction inspection is sufficient to assure this level of quality of construction.

2. The tabulated costs include the extra design and inspection costs required in earthquake prone areas, but that the normal costs of design and inspection have been provided for on a professional basis for comparable non-earthquake resistant structures.

3. The building including seismic resistant provisions is of comparable quality, material, durability, fire

TABLE 1: ESTIMATED INCREASED COSTS TO PROVIDE EARTHQUAKE RESISTANCE IN STRUCTURES [Adopted from Ref.(5)]

Area (Zone) / Type of Building	Areas which now enforce design for hurricane, cyclone, tornado or abnormally high winds	Other U.S. areas to meet Zone 3 requirements	Other U.S. areas located in Zones 0,1&2 to provide minimum requirements
1 & 2 storey Wood frame	1/2%	2%	1%
1,2,3 storey Brick or conc.block	4%	8%	4%
4 storey & up Brick or conc. block	5%	10%	5%
Concrete	2%	5%	2%
Steel frame	1/2%	3%	1%

resistance, etc. as the building without seismic provisions.

4. The increased cost in percentage is to be applied to the complete engineering and architectural building, including structure, foundation, walls, architectural treatment, mechanical and electrical facilities etc. It does not apply to site work such as streets, sidewalks, paving, landscaping, drainage, etc. nor does it apply to tenants' improvements.

With the above assumptions and the data given in Table 1, the average estimated increased cost of design and construction of any facility or group of facilities for the provisions of earthquake resistance can be determined in American conditions. The report also highlighted that the extra cost of design is an appreciable portion of the total increase since more analysis, drafting, detailing and field inspections are customarily required in seismic areas.

3. PRESENT WORK :

Three buildings have been chosen for the investigation in this work. These are all real structures in either completion, construction or planning stages. One is a four-storey institutional building, other an eight-storey residential building and the third a three-storey industrial building. These form a representative group of three different classes of practical structures. All the three buildings are of reinforced concrete framed construction and do not have shear walls. Seismic forces in each building have been obtained using seismic coefficient method and the building designed for different levels of loading and zones.

The following four load cases and corresponding designs have been examined in this study for each building.

(i) Only dead and live loads.

(ii) Vertical loads and seismic forces for O.C.F (K=1.0), i.e., equivalent to design conforming to IS:1893-1975.

(iii) Vertical loads and seismic forces for D.C.F (K=1.0).

(iv) Vertical loads and seismic forces for O.C.F (K=1.6).

All the above cases have been examined in Zones I, III and V for all the three buildings. The results obtained can be interpolated for zones II and IV. For each of the above load cases in each zone the quantities and cost of construction have been evaluated and a comparative study made.

Load Case (ii) has been included to show the effect of recent introduction of performance factor on the cost of construction. The premium for seismic resistance has been calculated over and above the cost of structure with only dead and live loads applied but not including wind loads. This study does not account for the additional cost involved in analysis, drafting, detailing and field inspections for D.C.F due to non-availability of reliable data on these costs. Ductility provisions of IS:4326 will mainly influence design of columns and beams. So, their effect on design of slabs, foundations and other structural and non-structural members has not been considered in this study.

4. ANALYSIS AND DESIGN :

Three dimensional linear elastic analysis of the buildings considered has been done using computer package

ETABS (Ref.6). Design and detailing of members has been done using design programs developed exclusively for this study and retaining the section sizes given by the original designer to the extent possible.

ETABS is a special purpose analysis package for building systems. With proper modelling of the structure, the program treats entire structure as three dimensional frame. It ignores axial deformation of beams and assumes floors to be rigid in their own plane. It takes care of rigid zones at ends of columns and beams and outputs member forces at the centre lines or at beam and column faces. It can also include shear deformations of members. There is provision for analysis of structures with shear walls and bracings. Detailed description and theoretical background of the program is given in Ref.(6). In this study the buildings considered do not have shear walls or bracings. The shear deformation of columns and beams have not been considered.

Analysis of each of the buildings was done for the following four loads cases (i) only dead loads (DL) (ii) only live loads (LL) (iii) forces due to earthquake in X-direction (ELX) (iv) forces due to earthquake in Y-direction (ELY). With member forces for these four load cases, following nine load combinations as recommended by IS:456 for limit state method, are generated within each of the design programs.

(a) 1.5 (DL + LL)
(b) 1.5 (DL + ELX)
(c) 1.5 (DL - ELX)
(d) 1.5 (DL + ELY)
(e) 1.5 (DL - ELY)
(f) 1.2 (DL + LL + ELX)

(g) 1.2 (DL + LL - ELX)

(h) 1.2 (DL + LL + ELY)

(i) 1.2 (DL + LL - ELY)

Four design programs have been developed in FORTRAN language to design the members by limit state method (Ref.4) using outputs of ETABS directly and to detail the members. These four programs have been named BEAM1, BEAM2, COLUM1 and COLUM2. BEAM1 and COLUM1 design flexural members and columns, respectively, for O.C.F. BEAM2 and COLUM2 design flexural members and columns, respectively, for D.C.F. These programs also calculate quantity of main and transverse steel using centre line lengths of members and sectional dimensions, but ignoring laps, anchorage lengths, wastages, etc. For each member, the programs also write information on governing load case (from among the nine load cases listed above), for both main steel and transverse steel. The programs for D.C.F write if the provisions of IS:4326 govern in design of tension, compression or transverse steel separately for each member. The members being rectangular in all the three buildings investigated, the programs have been developed only for rectangular members. In case of BEAM1 and BEAM2, the flange effect of slabs (i.e., T-Beam action) has been ignored.

For design shear forces and bending moments generated within the program for each member, BEAM1 calculates area of tension steel, compression steel and shear reinforcement required. Shear reinforcement is obtained for shear at a distance d (effective depth) from face of the column. It checks for minimum limits on steel as specified by IS:456. For detailing main reinforcement in beams, this program uses

available diameters of steel ranging from 12 mm to 28 mm and provides a set of arrangement that is closest to theoretical area of steel required. If two members are on either side of a column and are continuous in alignment, the program takes care of providing same reinforcement on both sides of the column by picking up the higher steel area between the two sections. For transverse reinforcement, the program provides a minimum diameter of 6 mm (even though a minimum diameter of 8 mm is desirable, at least in D.C.F). With the above detailing method, steel quantities obtained are fairly close to those for practical detailing. This program was used for flexural members in O.C.F.

In addition to detailing of beams as in BEAM1 above, BEAM2 checks for minimum and maximum reinforcement requirements of IS:4326 and finalizes main reinforcement in beams. With area of steel actually provided, it calculates negative and positive plastic moment capacities of sections at ends of each beam following the procedure outlined in Appendix :A of Part I. It considers strain hardening in main reinforcement by using tensile stress as 1.25 times the yield stress of steel and with no partial safety factor for material, γ_m as mentioned in Part I. With plastic moments calculated, the program calculates design shear forces at ultimate load condition, (i.e., loading condition corresponding to the stage at which plastic hinges are formed, as shown in Fig.5 of Part I) and designs for the governing shear force. It checks the maximum spacing requirements of IS:4326 for transverse steel. This program has been used for design and detailing of flexural members in D.C.F.

within the program for column elements, COLUM1 chooses a steel arrangement corresponding to minimum area of steel from within its library of arrangements, after checking adequacy for biaxial bending by generating interaction curves for a section. SP:16(S&T)-1980 (Ref.8) has given interaction curves by approximating sections to be having twenty bars, equally distributed on all four sides of sections. However, in COLUM1, interaction curves are generated for each section separately for each steel arrangement, making the design more exact and economical for design of columns. It designs transverse steel as per IS:456 using bar diameter of 6 mm as a minimum. This was used for design of columns in O.C.F.

In addition to design and detailing of columns as in COLUM1 above, COLUM2 calculates confinement steel as required by IS:4326 for rectangular sections and provides the same over a length at the ends of columns. This program was used for design and detailing of columns in D.C.F.

5. ASSUMED COST OF R.C. CONSTRUCTION :

To assess the difference in cost of structural framing systems, approximate current market rates of reinforced concrete construction have been assumed. The assumed cost of M15 concrete is Rs 750.00 per cubic metre, Fe415 steel reinforcement is Rs 8,000.00 per ton and shuttering for concrete is Rs 60.00 per square metre. The cost of concrete skeletons (only columns and beams) has been given as the overall cost, without consideration of cost of slabs, foundations, other structural and non-structural elements, and variation of unit rates with vertical lead.

6. FOUR-STOREY INSTITUTIONAL BUILDING :

6.1 BUILDING DESCRIPTION :

This is an educational building with lecture halls and laboratories. It has a base dimension of 15.8 m by 38.4 m and rises to a height of 16.25 m. The structural system consists of thirteen frames in transverse direction and four frames in longitudinal direction. The building has 100 mm thick solid slab at roof level and 120mm thick solid slabs at other floor levels. The foundation system consists of isolated footings. Figs.(1,2) show details of the building. The load class 400 of IS:875 (Ref.7) has been used in the lecture halls and laboratories, and load class 300 has been used in the cabins and corridors. The total dead load of structure is about 2500 tons and live load is about 400 tons.

6.2 RESULTS AND DISCUSSION:

Two complete sets of design results have been presented for this building. In Set 1 of designs, the sizes of the column sections of the original design could not accommodate steel within six percent for O.C.F(K=1.6) in zone V. So the column sizes were increased by about 40% for that particular case only. In Set 2, column sizes were so chosen that in zone I the percentage of steel was not too low and in zone V it does not cross six percent. This provided a simple basis for comparison, with variation limited to only the quantity of steel resulting from the designs. Results of the study have been summarised in Fig.(3) and Tables (2) to (6).

(i) ZONE I: For Set 1, D.C.F involves 2.50 ton more steel (9.6% higher) than O.C.F(K=1.6). The difference between

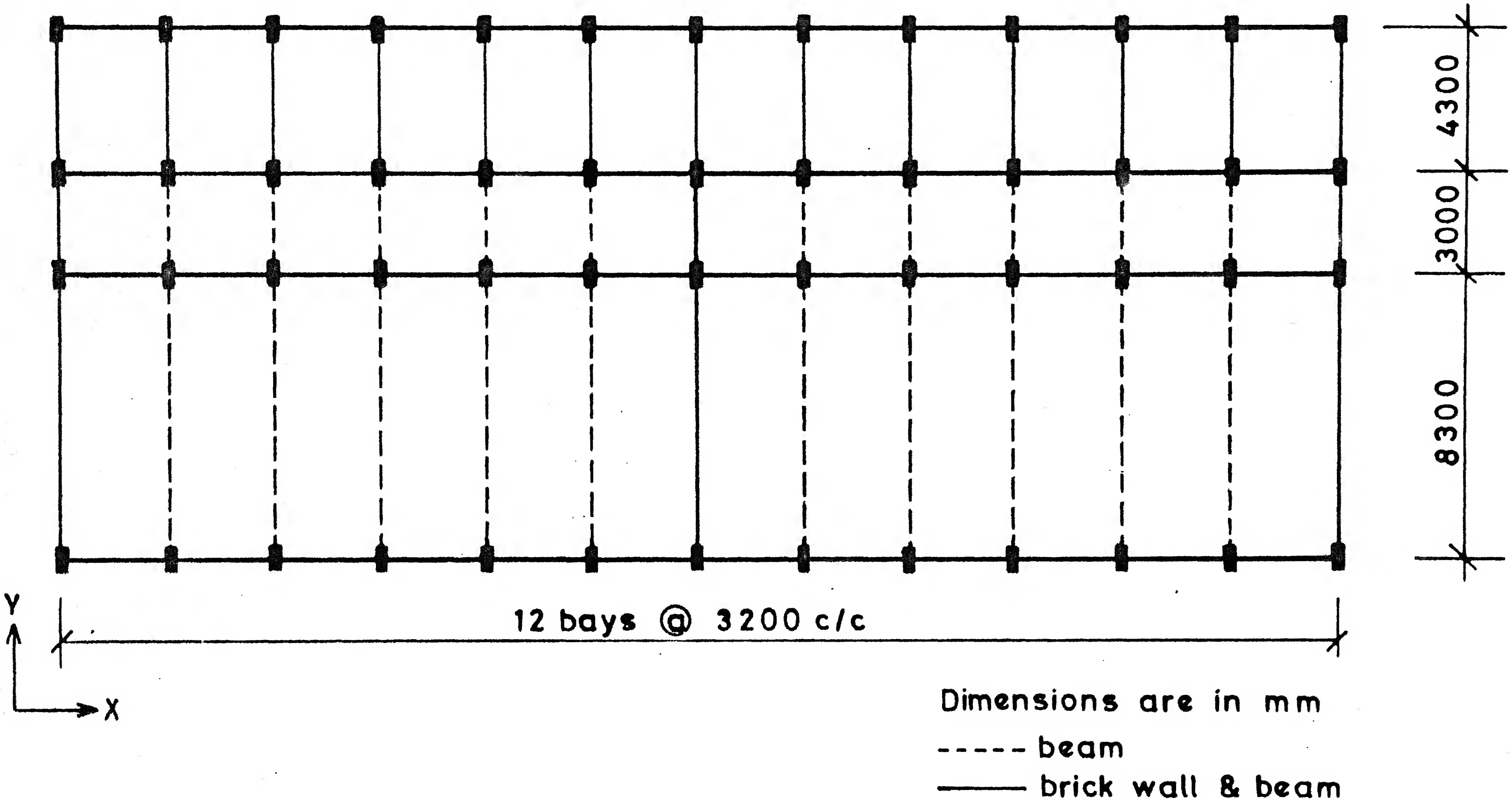
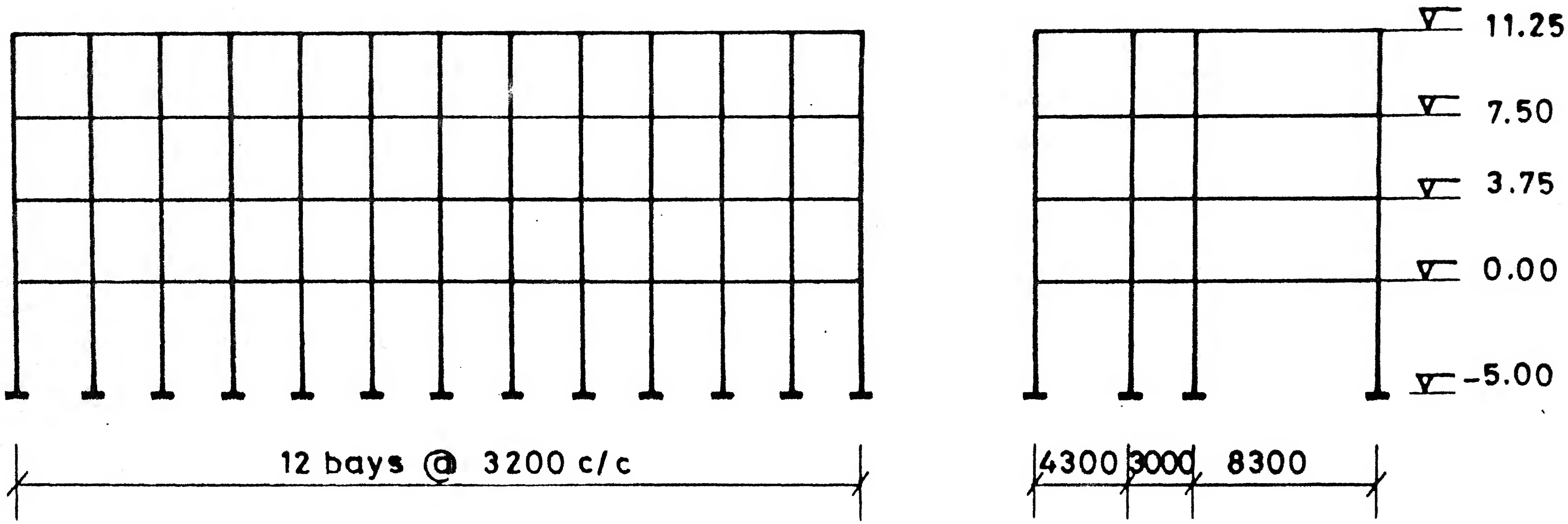


FIGURE 1 FLOOR PLAN FOR THE INSTITUTIONAL BUILDING.

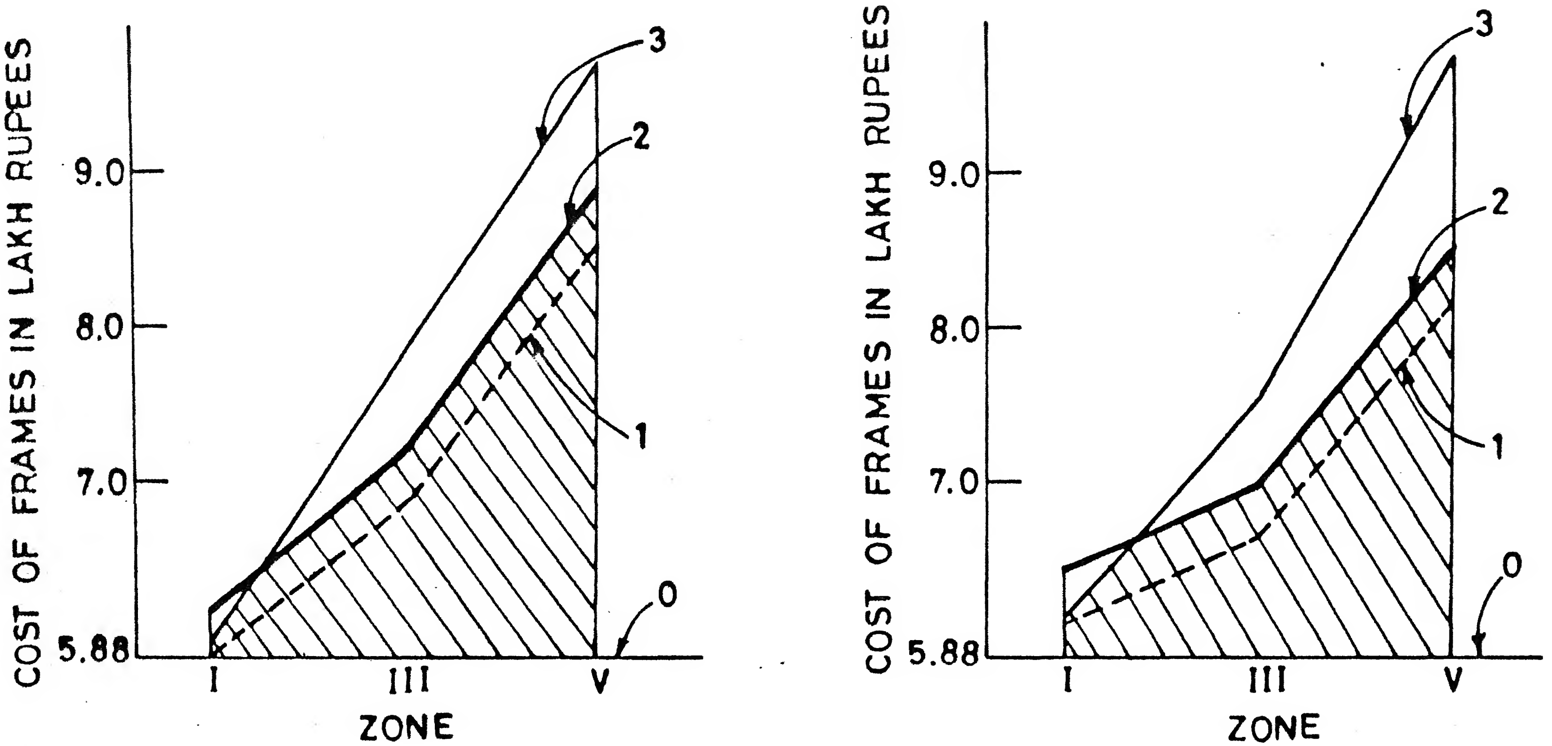


(a) Longitudinal frame

(b) Transverse frame

dimensions are in mm
levels are in meter

FIGURE 2 STRUCTURAL SYSTEM OF THE INSTITUTIONAL BUILDING



(a) Design with different column sections in different zones

(b) Design with same column sections in all zones

0 — COST OF CONCRETE FRAMES FOR DEAD AND LIVE LOADS

1 — COST OF ORDINARY CONCRETE FRAMES (K = 1.0)

2 — COST OF DUCTILE CONCRETE FRAMES (K = 1.0)

3 — COST OF ORDINARY CONCRETE FRAMES (K = 1.6)

FIGURE 3 COMPARISON OF COST OF SKELETON IN DIFFERENT ZONES FOR THE INSTITUTIONAL BUILDING.

TABLE 2: COMPARATIVE QUANTITIES OF STEEL FOR DESIGN WITH DIFFERENT COLUMN SECTIONS IN DIFFERENT ZONES(SET:1) : THE INTITUTIONAL BUILDING

		D.C.F. (K=1.0) (1)	D.C.F. (K=1.0) (2)	D.C.F. (K=1.6) (3)	Difference (2)over(1)		(3)over(1)		(3)over(2)	
		tons	tons	tons	tons	%	tons	%	tons	%
					Zone I					
Beams	Main	11.30	11.44	11.59	0.14	1.2	0.29	2.6	0.15	1.3
	Hoop	2.38	3.93	2.38	1.55	65.0	0.00	0.0	-1.55	-39.4
	Comb.	13.68	15.37	13.97	1.69	12.4	0.29	2.2	-1.40	-10.0
Colu-mns	Main	8.07	8.07	8.36	0.00	0.0	0.29	3.6	0.29	3.6
	Hoop	1.09	2.48	1.09	1.39	127.5	0.00	0.0	-1.39	-127.5
	Comb	9.16	10.55	9.45	1.39	15.2	0.29	3.2	-1.10	-10.4
	Total:	22.84	25.92	23.42	3.08	13.5	0.58	2.5	-2.50	-9.6
					Zone III					
Beams	Main	13.19	13.19	15.22	0.00	0.0	2.03	15.4	2.03	15.4
	Hoop	2.38	3.94	2.44	1.56	65.5	0.06	2.5	-1.50	-38.1
	Comb.	15.57	17.13	17.66	1.56	10.0	2.09	13.4	0.53	3.1
Colu-mns	Main	17.61	17.61	28.23	0.00	0.0	10.62	60.3	10.62	60.3
	Hoop	1.56	3.15	1.97	1.59	101.9	0.41	26.3	-1.18	-59.9
	Comb.	19.17	20.76	30.20	1.59	8.3	11.02	57.5	9.43	45.5
	Total:	34.74	37.94	47.85	3.20	9.2	13.10	37.7	9.91	26.1

TABLE 3: COMPARATIVE COST OF THE CONCRETE FRAME IN ZONE V FOR DESIGN WITH DIFFERENT COLUMN SECTIONS IN DIFFERENT ZONES (SET:1) : THE INSTITUTIONAL BUILDING

	O.C.F. (K=1.0) (1)	D.C.F. (K=1.0) (2)	O.C.F. (K=1.6) (3)	Difference					
				(2)over(1)		(3)over(1)		(3)over(2)	
	Lakh,Rs.	Lakh,Rs.	Lakh,Rs.	Lakh,Rs.	%	Lakh,Rs.	%	Lakh,Rs.	%
Beams	4.182	4.363	4.518	0.181	4.3	0.336	8.0	0.155	3.6
Columns	4.422	4.525	5.212	0.103	2.3	0.790	17.9	0.687	15.2
Total:	8.604	8.888	9.730	0.284	3.3	1.126	13.1	0.842	9.5

TABLE 4: COMPARATIVE QUANTITIES OF STEEL FOR DESIGN WITH SAME COLUMN SIZES IN ALL ZONES(SET:2) : THE INSTITUTIONAL BUILDING

		O.C.F. (K=1.0) (1)	D.C.F. (K=1.0) (2)	O.C.F. (K=1.6) (3)	Difference (2)over(1)		(3)over(1)		(3)over(2)	
		tons	tons	tons	tons	%	tons	%	tons	%
Zone I										
Beams	Main	11.30	11.44	11.59	0.14	1.2	0.29	2.6	0.15	1.3
	Hoop	2.38	3.93	2.38	1.55	65.0	0.00	0.0	-1.55	-39.4
	Comb.	13.68	15.37	13.97	1.69	12.4	0.29	2.2	-1.40	-10.0
Colu-mns	Main	6.36	6.36	6.60	0.00	0.0	0.24	3.8	0.24	3.8
	Hoop	1.13	3.33	1.13	2.20	194.7	0.00	0.0	-2.20	-194.7
	Comb.	7.49	9.69	7.73	2.20	29.4	0.24	3.2	-1.96	-20.2
	Total:	21.16	25.06	21.70	3.90	18.4	0.54	2.6	-3.36	-13.4
Zone III										
Beams	Main	13.19	13.19	15.22	0.00	0.0	2.03	15.4	2.03	15.4
	Hoop	2.38	3.94	2.44	1.56	65.6	0.06	2.5	-1.50	-38.1
	Comb.	15.57	17.13	17.66	1.56	10.0	2.09	13.4	0.53	3.1
Colu-mns	Main	11.63	11.63	20.07	0.00	0.0	8.44	72.6	8.44	72.6
	Hoop	1.23	3.41	1.84	2.18	177.2	0.61	49.6	-1.57	-46.0
	Comb.	12.86	15.04	21.91	2.18	17.0	9.05	70.4	6.87	45.7
	Total:	28.43	32.17	39.57	3.74	13.2	11.14	39.2	7.40	23.0

Table 4 (continued)

		O.C.F. (K=1.0) (1)	D.C.F. (K=1.0) (2)	O.C.F. (K=1.6) (3)	Difference (2)over(1)		(3)over(1)		(3)over(2)	
		tons	tons	tons	tons	%	tons	%	tons	%
Zone V										
Beams	Main	16.43	17.00	20.03	0.57	3.5	3.60	21.9	3.03	17.8
	Hoop	2.58	4.25	3.16	1.67	64.7	0.58	22.5	-1.09	-25.7
	Comb.	19.01	21.25	23.19	2.24	11.8	4.18	22.0	1.94	9.1
Colu-mns	Main	25.96	25.96	42.43	0.00	0.0	16.47	63.4	16.47	63.4
	Hoop	2.02	3.60	3.08	1.58	78.2	1.06	52.5	-0.52	-14.4
	Comb.	27.98	29.56	45.51	1.58	5.7	17.53	62.7	15.95	54.0
	Total:	46.99	50.81	68.7	3.82	8.2	21.71	46.2	17.89	35.2

TABLE 5: COMPARATIVE COSTS FOR DIFFERENT ZONES : THE INSTITUTIONAL BUILDING

Cost in lakh Rs.	DL+LL only (0)	D.C.F. (K=1.0) (1)	D.C.F. (K=1.0) (2)	D.C.F. (K=1.6) (3)	Difference					
					(2)over(1)		(3)over(1)		(3)over(2)	
Zone					Cost	%	Cost	%	Cost	%
FOR DESIGN WITH DIFFERENT COLUMN SECTIONS IN DIFFERENT ZONES (SET: 1)										
I	5.88	5.89	6.16	6.00	0.27	4.6	0.11	1.9	-0.16	-2.6
III	5.88	6.88	7.16	7.88	0.28	4.1	1.00	14.5	0.72	10.1
V	5.88	8.60	8.89	9.73	0.29	3.4	1.13	13.1	0.84	9.5
FOR DESIGN WITH SAME COLUMN SIZES IN ALL ZONES (SET: 2)										
I	5.88	6.08	6.39	6.12	0.31	4.9	0.04	0.7	-0.27	-4.2
III	5.88	6.66	6.96	7.55	0.30	4.3	0.89	13.4	0.59	8.5
										6.7

TABLE 6: PREMIMUM FOR EARTHQUAKE RESISTANCE : THE INSTITUTIONAL BUILDING)

Zone	Premium in Rs. per square metre per floor		
	O.C.F (K=1.0)	D.C.F (K=1.0)	O.C.F (K=1.6)
DESIGN WITH DIFFERENT COLUMN SECTIONS IN DIFFERENT ZONES (SET 1)			
I	0.50	12.00	5.00
III	42.00	53.00	84.00
V	114.00	126.00	161.00
DESIGN WITH SAME COLUMN SECTIONS IN ALL ZONES (SET 2)			
I	8.00	21.00	10.00
III	33.00	45.00	70.00
V	94.00	107.00	161.00

O.C.F(K=1.6) and O.C.F(K=1.0) is 0.58 tons (25%), D.C.F involves Rs 0.16 lakhs more cost than O.C.F(K=1.6) which amounts to 2.6 percent of the total cost of skeleton. The difference of cost between O.C.F(K=1.6) and O.C.F(K=1.0) is about Rs 0.11 lakhs, (1.9%). The premium of seismic resistance is about Rs 5.00 per square metre of plinth area per floor. Set 2 is a more expensive alternative for the building in Zone I and its results show the same trend as those for Set 1.

(ii) ZONE III: Steel in D.C.F is 9.91 tons less (26.1%) than that in O.C.F(K=1.6) for Set 1 and 7.40 tons less (23.0%) in Set 2. O.C.F(K=1.6) involves 13.10 tons more (37.7%) steel for Set 1 and 11.14 tons more (39.2%) steel for Set 2, than O.C.F(K=1.0). D.C.F is cheaper by Rs 0.72 lakhs, i.e. 10.1% as compared to the cost of O.C.F(K=1.6). Cost of O.C.F (K=1.6) is about Rs 1 lakh more (14.5%) than that of O.C.F (K=1.0). The difference between D.C.F and O.C.F(K=1.0) is Rs 0.28 lakhs, (4.1%). The premium for seismic resistance in zone III is about Rs 45.00 to Rs 53.00 per square metre of plinth area per floor, if D.C.F is provided and about Rs 70.00 to Rs 84.00 per square metre if O.C.F(K=1.6) is used.

(iii) ZONE V: By providing D.C.F a saving of 17.89 tons of steel (35.2%) is achieved over O.C.F(K=1.6) for Set 2. O.C.F(K=1.6) involves 21.71 tons (46.2%) more steel than O.C.F(K=1.0) in Set 2. D.C.F is cheaper than O.C.F(K=1.6) by Rs 0.84 lakhs (9.5%) and costlier than O.C.F(K=1.0) by Rs 0.29 lakhs (3.4%). This difference is Rs 1.43 lakhs, (16.7%) and Rs 0.31 lakhs (3.8%) for Set 2. Premium per square metre of plinth area per floor for seismic resistance

in Zone V with D.C.F is Rs 145.00 for Set 1 and Rs 107.00 for Set 2. Corresponding premium is about Rs 161.00 if O.C.F(K=1.6) is used. It is thus seen that the premium largely depends on the column sizes chosen.

Some of the important inferences from these results are listed below:

(a) In Zone I, providing D.C.F is uneconomical and the premium of earthquake resistance is very low. The difference between O.C.F(K=1.0) and O.C.F(K=1.6) is negligible.

(b) For Zone III, difference in cost between D.C.F and O.C.F(K=1.6) is nominal, however D.C.F is cheaper. There is remarkable difference between O.C.F(K=1.6) and O.C.F (K=1.0).

(c) For Zone V, D.C.F is distinctly cheaper than O.C.F (K=1.6). Use of O.C.F(K=1.6) involves large extra cost over O.C.F(K=1.0) and D.C.F. Selection of section sizes greatly influences the premium for earthquake resistance.

7. EIGHT-STOREY RESIDENTIAL BUILDING :

7.1 BUILDING DESCRIPTION:

This is an H-shaped residential building. A structural separation joint has been provided (Fig.4) to avoid the interaction of central core with each of the wings. One wing of the building has been analyzed in this study. It has a base dimension of 5.20 m by 21.6 m and rises to a height of 25.65 m (Figs.4,5). The structural system for each wing consists of seven frames in transverse direction and two frames in longitudinal direction. The floors consist of solid slabs ranging from 90 mm to 130 mm in thickness. The

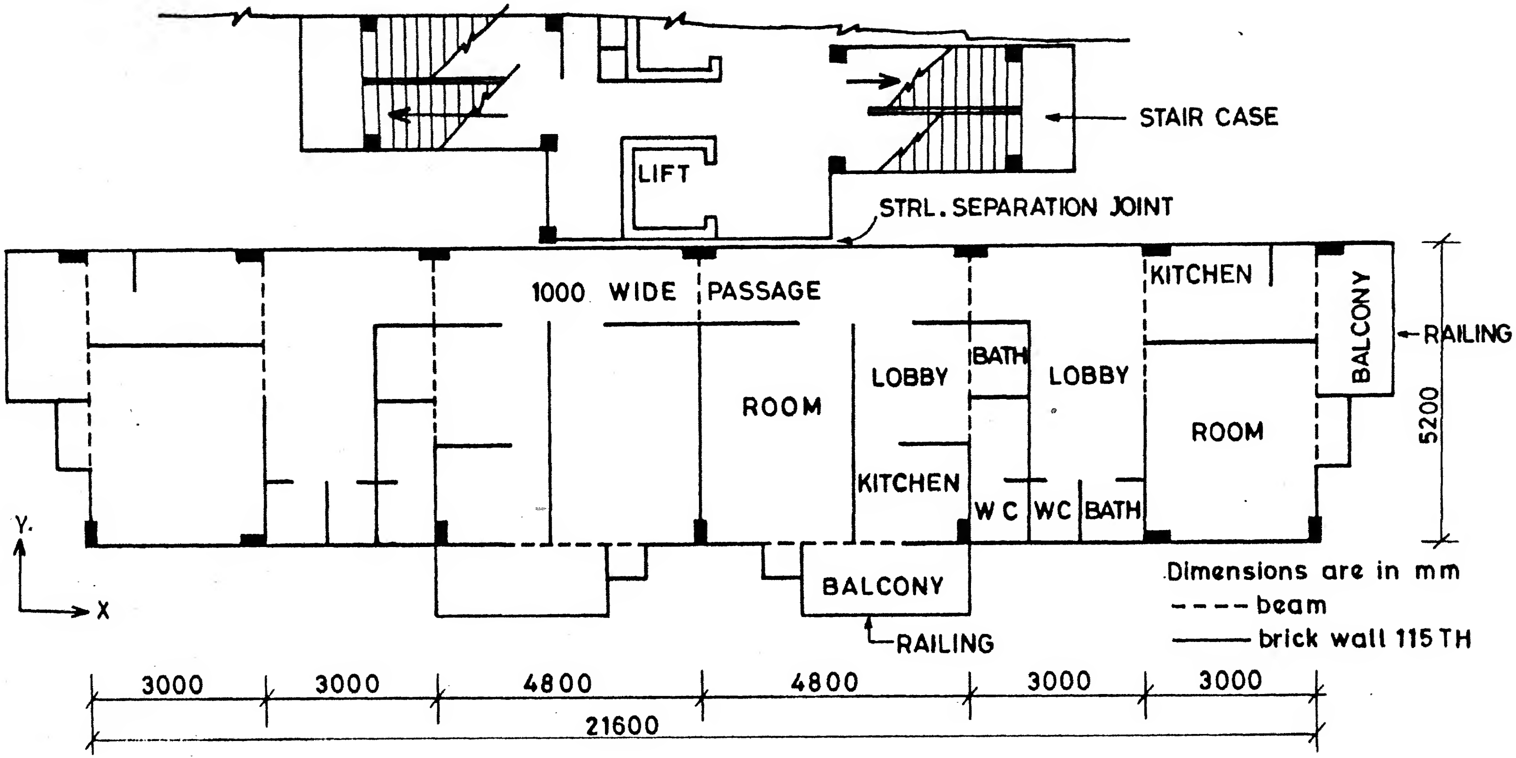
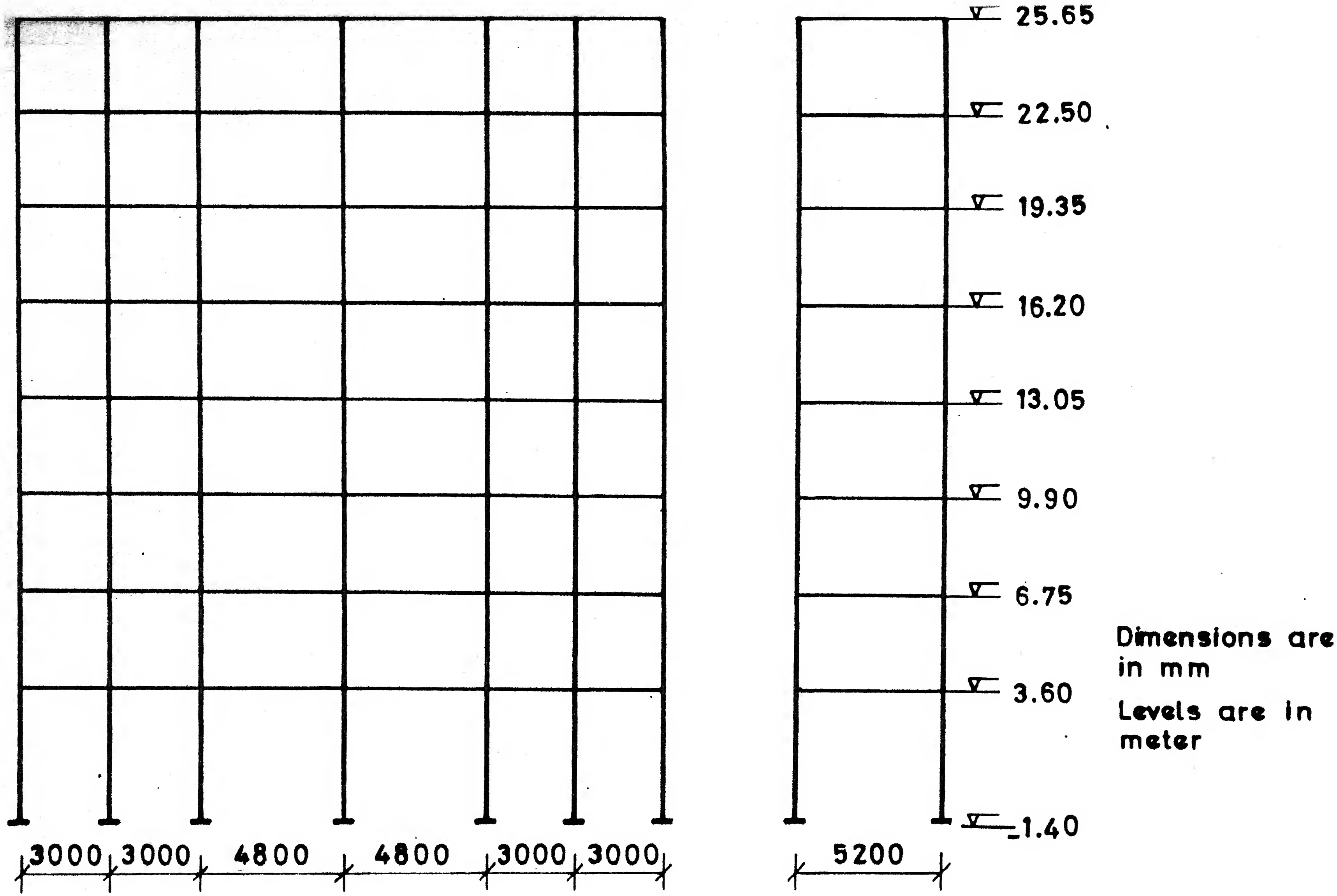


FIGURE 4 FLOOR PLAN FOR THE RESIDENTIAL BUILDING.



(a) Longitudinal Frame

(b) Transverse Frame

FIGURE 5 STRUCTURAL SYSTEM FOR THE RESIDENTIAL BUILDING.

a residential building, load class 200 of IS:875 has been used throughout the building. 115 mm thick brick walls have been provided by the architect for all partitions and external walls. With thin slabs, half brick walls and 3.15 m of inter-storey height, this structure is a very lightly loaded structure. Total dead load of the structure is about 1300 tons and total live load is about 175 tons.

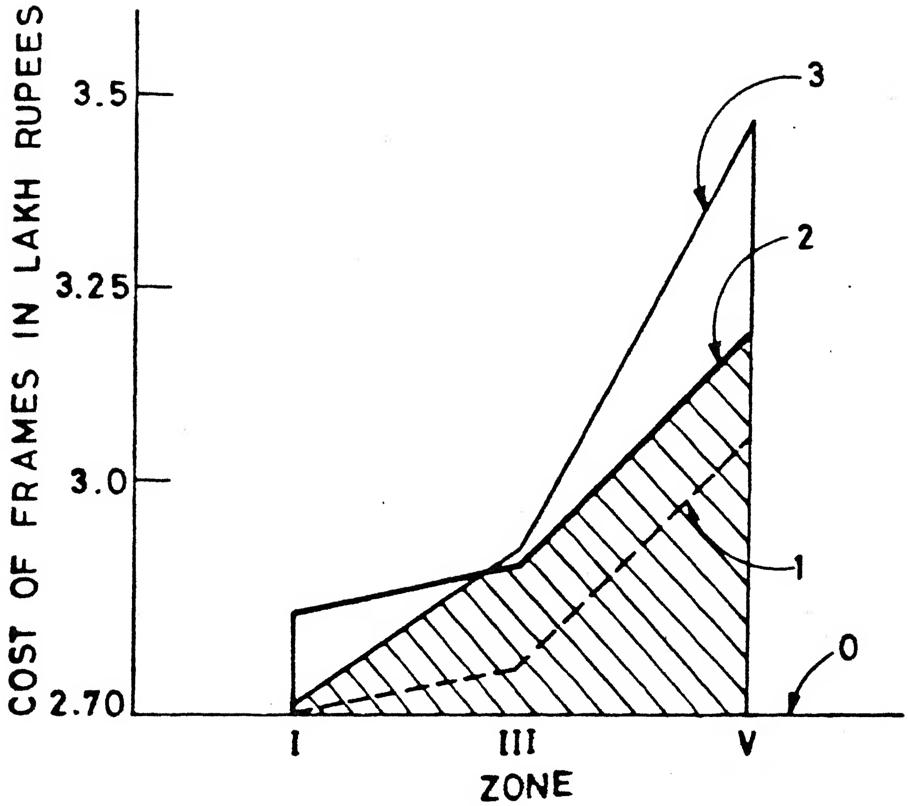
7.2 RESULTS AND DISCUSSION:

Results for this building have been summarised in Fig.(6) and Table (7,8).

(i) ZONE I: D.C.F involves 1.45 tons (12.6%) more steel as compared to O.C.F(K=1.6) and 0.14 tons (1.4%) more steel is incurred if O.C.F(K=1.6) is provided in place of O.C.F (K=1.6). It is costlier by Rs 0.12 lakhs (4.1%) to provide D.C.F as compared to O.C.F(K=1.6). The premium for earthquake resistance is about Rs 2.00 per square meter of plinth area per floor.

(ii) ZONE III: Difference in quantity for D.C.F and O.C.F (K=1.6) is very nominal. It comes out to be 0.24 tons less (2%) steel in D.C.F as compared to O.C.F(K=1.6). O.C.F (K=1.6) involves about 1.85 tons (17.2%) more steel as compared to O.C.F (K=1.0). The difference in cost between D.C.F and O.C.F (K=1.6) is very nominal, however O.C.F (K=1.6) is costlier by Rs 0.15 lakhs (5.4%) as compared to O.C.F (K=1.0). The premium for earthquake resistance is about Rs 22.00 per square metre of plinth area per floor.

(iii) ZONE V: By providing D.C.F, 3.50 tons less (21.7%) steel is needed as compared to that for O.C.F(K=1.6). Use of O.C.F(K=1.6) increased the steel quantity by 5.16 tons



0 — COST OF CONCRETE FRAMES FOR DEAD AND LIVE LOADS

1 — COST OF ORDINARY CONCRETE FRAMES (K = 1.0)

2 — COST OF DUCTILE CONCRETE FRAMES (K = 1.0)

3 — COST OF ORDINARY CONCRETE FRAMES (K = 1.6)

FIGURE 6 COMPARISION OF COST OF SKELETON IN DIFFERENT ZONES FOR THE RESIDENTIAL BUILDING.

TABLE 7: COMPARATIVE QUANTITIES OF STEEL : THE RESIDENTIAL BUILDING

		O.C.F. (K=1.0) (1)	D.C.F. (K=1.0) (2)	O.C.F. (K=1.6) (3)	Difference (2)Over(1)		(3)Over(1)		(3)Over (2)	
		Tons	Tons	Tons	Tons	%	Tons	%	Tons	%
ZONE I										
Beam	Main	3.95	3.95	4.03	0.00	0.0	0.08	2.0	0.08	2.0
	Hoop	0.71	1.38	0.71	0.67	94.4	0.00	0.0	-0.67	-94.4
	Comb	4.66	5.33	4.74	0.67	14.4	0.08	1.7	-0.59	-11.1
Col-umns	Main	4.65	4.65	4.65	0.00	0.0	0.00	0.0	0.00	0.0
	Hoop	0.63	1.55	0.69	0.92	146.0	0.06	9.5	-0.86	-55.5
	Comb	5.28	6.20	5.34	0.92	17.4	0.06	1.1	-0.86	-13.9
	Total	9.94	11.53	10.08	1.59	16.0	0.14	1.4	-1.45	-12.6
ZONE III										
Beam	Main	4.42	4.49	4.92	0.07	1.6	0.50	11.3	0.43	9.6
	Hoop	0.72	1.40	0.74	0.68	94.4	0.02	2.8	-0.66	-47.0
	Comb	5.14	5.89	5.66	0.75	14.6	0.52	10.1	-0.23	-3.9
Col-umns	Main	4.89	4.89	6.19	0.00	0.0	1.30	26.6	1.30	26.6
	Hoop	0.70	1.56	0.73	0.86	122.9	0.08	4.3	-0.83	-53.2
	Comb	5.59	6.45	6.92	0.86	15.4	1.33	23.8	0.47	7.3
Total		10.73	12.34	12.58	1.61	15.0	1.85	17.2	0.24	2.0

Table 7 (continued)

		O.C.F. (K=1.0) (1)	D.C.F (K=1.0) (2)	O.C.F. (K=1.6) (3)	Difference					
					(2)Over(1)		(3)Over(1)		(3)Over (2)	
		Tons	Tons	Tons	Tons	%	Tons	%	Tons	%
ZONE V										
	Main	5.27	5.60	6.20	0.33	6.3	0.93	17.7	0.60	10.7
Beam	Hoop	0.77	1.45	1.18	0.68	46.9	0.41	53.3	-0.27	-18.6
	Comb	6.04	7.05	7.38	1.01	16.7	1.34	22.2	0.33	4.7
	Main	7.38	7.38	11.20	0.00	0.0	3.82	51.8	38.20	51.8
Col-	Hoop	1.02	1.67	1.02	0.65	63.7	0.00	0.0	-0.65	-38.9
umns	Comb	8.40	9.05	12.22	0.65	7.7	3.82	45.5	3.17	35.0
Total		14.44	16.10	19.60	1.66	11.5	5.16	35.7	3.50	21.7

TABLE 8. COMPARATIVE COSTS FOR DIFFERENT ZONES AND PREMIUM FOR EARTHQUAKE RESISTANCE : THE RESIDENTIAL BUILDING

(a) COMPARISION OF COSTS:

Cost in lakh Rs	DL+LL only	O.C.F. (K=1.0)	D.C.F. (K=1.0)	O.C.F. (K=1.6)	Difference					
					(2)over(1)		(3)over(1)		(3)over(2)	
Zone	(0)	(1)	(2)	(3)	Cost	%	Cost	%	Cost	%
I	2.70	2.70	2.83	2.71	0.13	4.7	0.01	0.4	-0.12	-4.1
III	2.70	2.76	2.89	2.91	0.13	4.7	0.15	5.4	0.02	0.7
V.	2.70	3.06	3.19	3.47	0.13	4.3	0.41	13.5	0.28	8.8

(b) PREMIUM DETAILS:

	Premium in Rs per square metre per floor		
Zone	O.C.F (K=1.0)	D.C.F (K=1.0)	O.C.F (K=1.6)
I	0.00	14.00	2.00
III	7.00	22.00	24.00
V	40.00	55.00	86.00

(35.7%) as compared to O.C.F(K=1.0). If financial implications are considered, D.C.F is cheaper by Rs 0.28 lakhs (8.8%) as compared to O.C.F(K=1.6). Premium per square metre per floor for earthquake resistance is about Rs 55.00 if D.C.F is provided and Rs 86.00 if O.C.F(K=1.6) is provided.

From the above results, it can be seen that providing D.C.F is uneconomical in zone I. Even though the difference is not remarkable, D.C.F is cheaper in Zone III. In Zone V there is significant saving by providing D.C.F.

8. THREE-STOREY INDUSTRIAL BUILDING :

8.1 BUILDING DESCRIPTION:

In this building the basement is a storage godown, the ground floor a machine room housing light machines which do not produce vibrational disturbance and first floor is an office space. Figs. (7,8) show the details of the building. The building has an intermediate storey, whose height is much more than that of the other storeys. Thus, this is an irregular building and it does not fulfill requirements for use of seismic coefficient method given in the code. In absence of overall lack of earthquake engineering background among many designers in the country such irregular buildings continue to be designed by seismic coefficient method. In fact, such buildings can be analyzed by seismic coefficient method under certain conditions (Ref.9).

The building has base dimensions of 42.0 m by 28.0 m and rises to a height of 14.15 m above foundation level. The structural system consists of four frames of 42.0 m length and a fifth frame of 35.0 m length in transverse direction and six frames of 28.0 m length and one of 21.0 m in

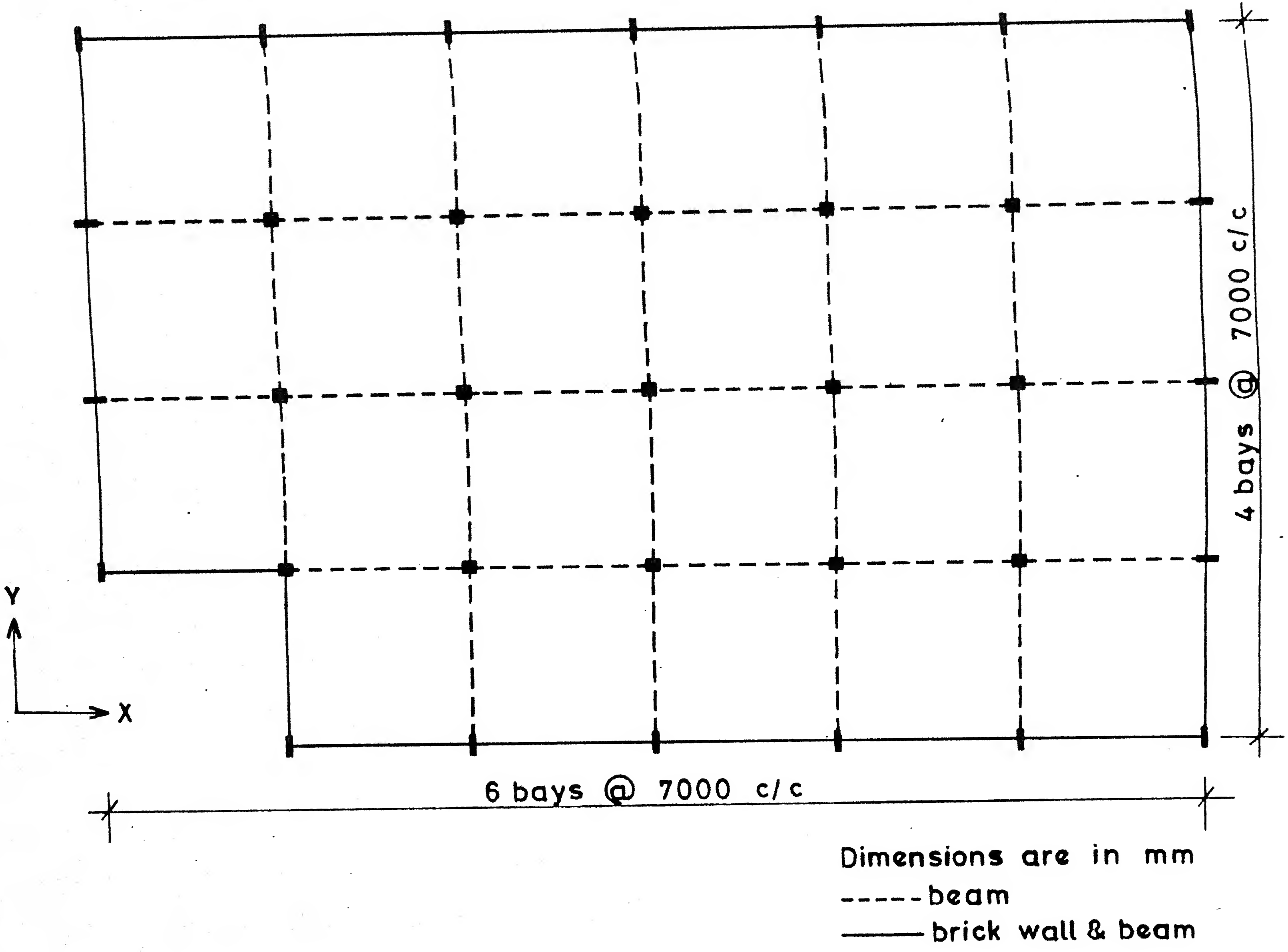


FIGURE 7 FLOOR PLAN FOR THE INDUSTRIAL BUILDING.

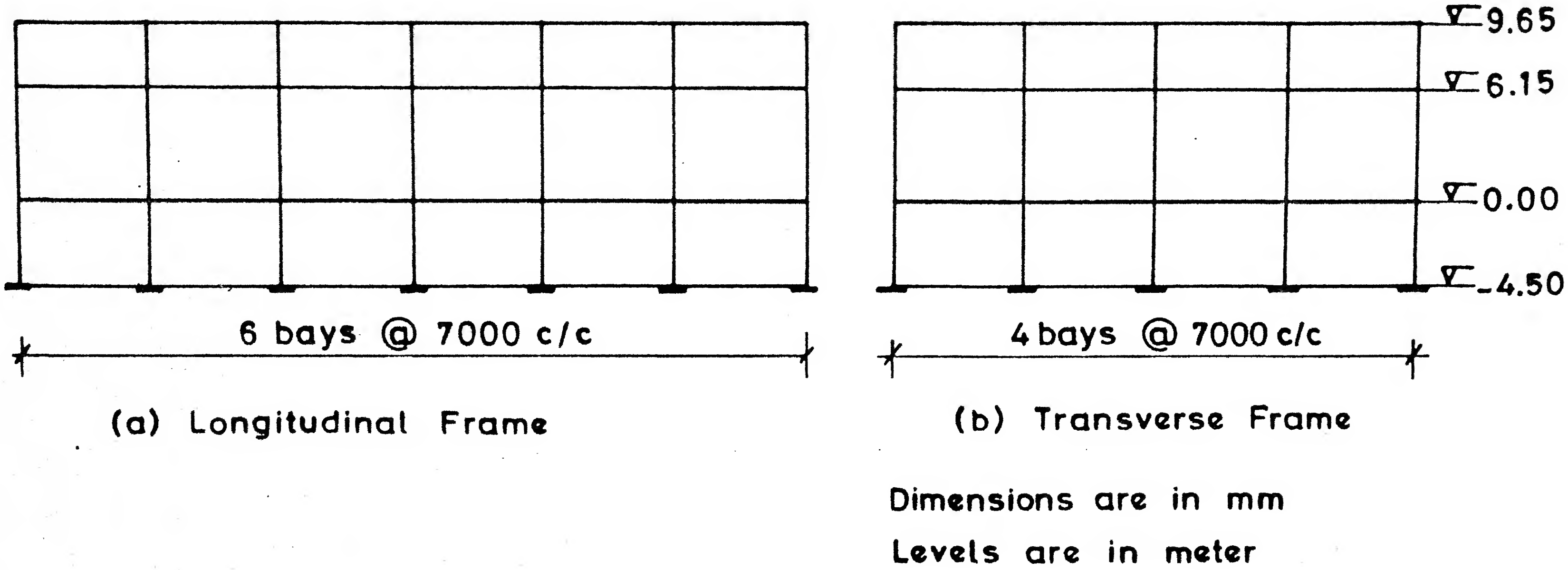


FIGURE 8 STRUCTURAL SYSTEM OF THE INDUSTRIAL BUILDING.

longitudinal direction. Loading of 10 KN per square metre, on ground floor, and 3 KN per square metre on first floor were used. Total dead load of the structure is about 2700 tons and live load is about 1500 tons.

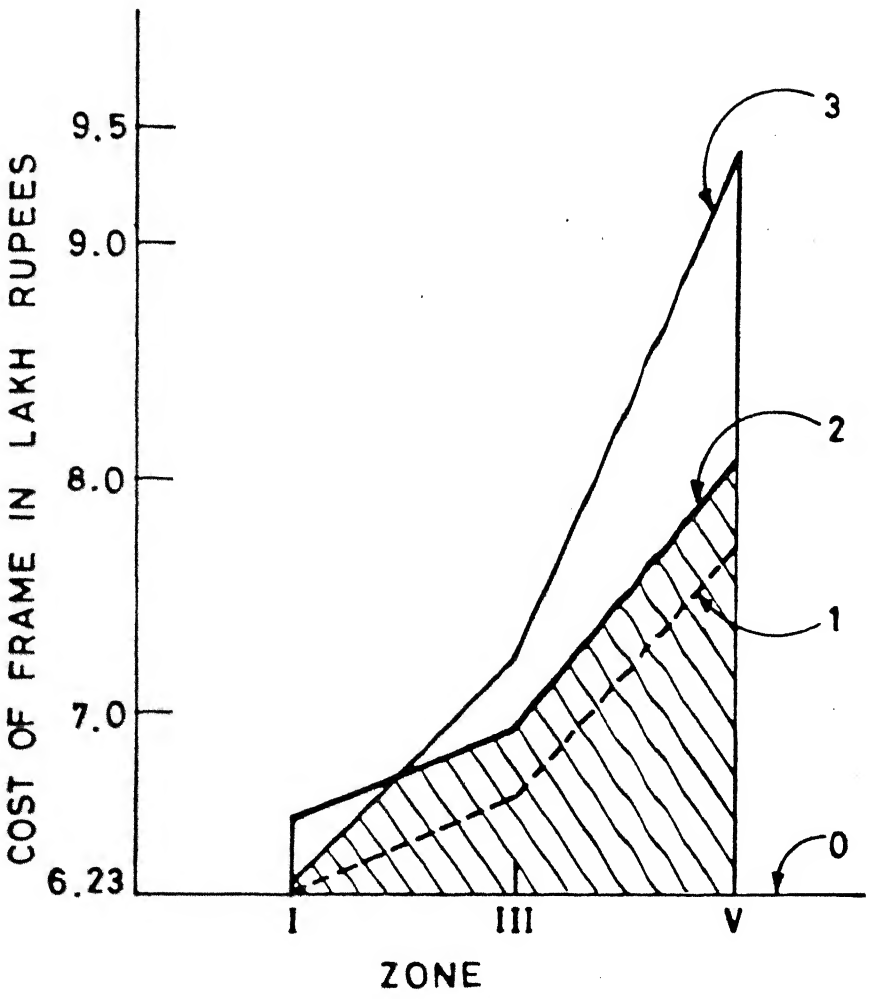
8.2 RESULTS AND DISCUSSION:

The results for this building have been summarised in Fig.(9) and tables (9,10).

(i) ZONE I: In zone I, D.C.F involves 3.43 tons (12.9%) more steel as compared to O.C.F(K=1.6). O.C.F(K=1.6) involves negligibly more amount of steel than is required by O.C.F(K=1.0). D.C.F becomes costlier by Rs 0.28 lakhs (4.3%) as compared to O.C.F(K=1.6). There is practically no difference in cost between O.C.F(K=1.6) and O.C.F(K=1.0). The premium for earthquake resistance in Zone I is negligible.

(ii) ZONE III: D.C.F involves 3.72 tons (11.8%) less steel than O.C.F(K=1.6), which in turn involves 7.30 tons (26.0%) more steel than O.C.F(K=1.0). D.C.F is cheaper by Rs 0.30 lakhs (4.3%) as compared to O.C.F(K=1.6). The difference between O.C.F(K=1.6) and O.C.F(K=1.0) is about Rs 0.58 lakhs (8.7%). Premium for earthquake resistance per square metre of plinth area per floor is about Rs 21.00 if D.C.F is used and about Rs 30.00 if O.C.F(K=1.6) is used.

(iii) ZONE V: D.C.F involves 16.99 tons (36.9%) less steel than O.C.F(K=1.6), which in turn requires 21.34 tons (51.2%) more steel as compared to O.C.F(K=1.0). D.C.F is cheaper by Rs 1.36 lakhs (16.8%) as compared to O.C.F(K=1.6). There is a difference of Rs 1.70 lakhs (22.8%) between O.C.F(K=1.6) and O.C.F(K=1.0). Premium for earthquake resistance per



0 — COST OF CONCRETE FRAMES FOR DEAD AND LIVE LOADS

1 — COST OF ORDINARY CONCRETE FRAMES (K = 1.0)

2 — COST OF DUCTILE CONCRETE FRAMES (K = 1.0)

3 — COST OF ORDINARY CONCRETE FRAMES (K = 1.6)

FIGURE 9 COMPARISION OF COST OF SKELETON IN DIFFERENT ZONES FOR THE INDUSTRIAL BUILDING.

TABLE 9: COMPARATIVE QUANTITIES OF STEEL : THE INDUSTRIAL BUILDING

		O.C.F. (K=1.0) (1)	D.C.F. (K=1.0) (2)	O.C.F. (K=1.6) (3)	Difference (2)over (1)		Difference (3)over (1)		Difference (3)over (2)	
		tons	tons	tons	tons	%	tons	%	tons	%
Zone I										
Beams	Main	12.58	12.79	12.62	0.21	1.7	0.04	0.3	-0.17	-1.3
	Hoops	3.03	5.24	3.03	2.21	73.0	0.00	0.0	-2.21	-73.0
	Comb	15.61	18.03	15.65	2.42	15.5	0.04	0.3	-2.38	-13.2
Col-umns	Main	6.60	6.60	6.82	0.00	0.0	0.22	3.3	0.22	3.3
	Hoops	0.79	2.06	0.79	1.27	160.8	0.00	0.0	-1.27	-160.8
	Comb	7.39	8.66	7.61	1.27	17.2	0.22	3.0	-1.05	-12.1
	Total	23.00	26.69	23.26	3.69	16.0	0.26	1.1	-3.43	-12.9
Zone III										
Beams	Main	14.06	14.16	15.81	0.11	0.8	1.76	12.5	1.65	11.7
	Hoops	3.03	5.32	3.04	2.29	75.6	0.01	0.3	-2.28	-42.9
	Comb	17.08	19.46	18.85	2.40	14.1	1.77	10.4	-0.63	-3.2
Col-umns	Main	10.13	10.13	15.50	0.00	0.0	5.37	53.0	5.37	53.0
	Hoops	0.85	2.03	1.01	1.18	138.8	0.16	18.8	1.02	50.3
	Comb	10.98	12.16	16.51	1.18	10.8	5.53	50.4	4.35	35.8
	Total	28.06	31.64	35.36	3.58	12.8	7.30	26.0	3.72	11.8

Table 9: (continued)

		O.C.F. (K=1.0) (1)	D.C.F. (K=1.0) (2)	O.C.F. (K=1.6) (3)	Difference (2)over (1)		(3)over (1)		(3)over (2)	
		tons	tons	tons	tons	%	tons	%	tons	%
					Zone V					
	Main	17.16	18.11	18.21	0.95	5.5	1.05	6.61	0.10	0.6
	Hoops	3.09	5.74	3.44	2.65	85.8	0.35	11.3	-2.30	-40.1
Beams	Comb	20.25	23.85	21.65	3.60	17.8	1.40	6.9	-2.20	-9.2
	Main	20.10	20.10	38.65	0.00	0.0	18.55	92.3	18.55	92.3
Col-	Hoops	1.32	2.07	2.71	0.75	56.8	1.39	105.3	0.64	30.9
umns	Comb	21.42	22.17	41.36	0.75	3.5	19.94	93.1	19.26	87.0
	Total	41.67	46.02	63.01	4.35	10.5	21.34	51.2	16.99	36.9

TABLE 10: COMPARATIVE COST FOR DIFFERENT ZONES AND PREMIUM FOR EARTHQUAKE RESISTANCE : THE INDUSTRIAL BUILDING

(a) COMPARISION OF COST:

Cost in lakh Rs	DL+LL	O.C.F. (K=1.0)	D.C.F. (K=1.0)	O.C.F. (K=1.6)	Difference (2)over(1)		Difference (3)over(1)		Difference (3)over(2)	
Zone	(0)	(1)	(2)	(3)	Cost	%	Cost	%	Cost	%
I	6.23	6.24	6.54	6.26	0.30	4.8	0.02	0.30	-0.28	-4.3
III	6.23	6.65	6.93	7.23	0.28	4.2	0.58	8.7	0.30	4.3
V	6.23	7.74	8.06	9.44	0.34	4.4	1.70	22.0	1.36	16.8

(b) PREMIUM DETAILS:

Zone	Premium in Rs per square metre per floor: O.C.F (K=1.0)	D.C.F (K=1.0)	O.C.F (K=1.6)
I	0.00	9.00	0.00
III	12.00	21.00	30.00
V	45.00	55.00	95.00

square metre of plinth area per floor is about Rs 55.00 if D.C.F is provided and Rs 95.00 if O.C.F(K=1.6) is provided.

The inference from above results is that, in Zone I D.C.F is costlier than O.C.F(K=1.6), in Zone III D.C.F is cheaper and, in Zone V D.C.F is significantly cheaper.

9. COMPARISION OF RESULTS :

CPWD recommends extra cost for providing earthquake resistance for non-residential buildings in R.C. framed construction as Rs 34.00 per square meter, as on 1.10.1976. This recommendation does not account for the seismic zone and seems to be based on constrution in New Delhi (zone IV). At current cost index it is about Rs 100.00 per square metre, but this extra cost seems to be for non-ductile construction. This type of construction is represented by the institutional building in this study, where the premium for earthquake resistance in Zone III is about Rs 43.00 to Rs 53.00 and corresponding premium in Zone V is about Rs 107.00 to Rs145.00 per square metre per floor, if ductile frames are used.

10. GENERAL :

The following general observations were made during the course of this work:

(i) In earthquake resistant design, for low values of live loads on the structure, the load condition 1.5(DL $\pm$ EL) usually governs the design. However, some of the members are governed by the load condition 1.2 (DL + LL + EL).

(ii) In beams of D.C.F, for normal sizes and loads (a) maximum limit of longitudinal reinforcement as specified by

IS:4326 occassionally governs the design (b) minimum longitudinal steel condition often governs at sections where only hanger bars are required (c) limit on maximum spacing (i.e., d/4 for distance 2d from face of column and d/2 for remaining length) for transverse steel almost always governs. Thus for moderately reinforced sections with normal spans, design of shear reinforcement with consideration of plastic moment capacities does not govern the design very often.

(iii) Proper choice of column sizes largely influences quantity of steel and cost of structures.

(iv) With judicious selection of member sizes in preliminary design, limits on maximum steel and minimum steel can be effectively avoided for flexural members and, for columns large biaxial moments can be economically taken care of. Flexural members should be aimed to be provided with depths close to balanced sections.

11. SUMMARY AND CONCLUSIONS :

With the introduction of performance factor, K, in the fourth edition of IS:1893, the cost of earthquake resistance has, in addition to other factors, become dependent on the ductility of construction adopted in the structure. Three practical buildings have been thoroughly analyzed and designed to study cost implication for ductile as well as non-ductile construction in zones 1, III and V. The cost of structures designed as per the earlier edition of code, i.e. IS:1893-1975, has also been shown. Besides, design for dead load and live load only has been done and premium for earthquake resistance for these buildings has been given. An experienced designer can judiciously utilize the percentage

increase in cost of structures and quantity of steel reported here, in planning and preliminary design stages of similar structures. Since no two designers are likely to arrive at the same final design and detailing, the figures given in this study may vary considerably but these may be treated as rough guidelines for similar structures.

The three buildings considered belong to three different classes of building and have different number of storeys, but the trend of results has been quite similar. Following broad conclusions can be drawn from the results.

(i) Introduction of performance factor, K, in the fourth edition of IS:1893 has distinctly increased cost of structures not detailed as per IS:4326. As per requirements of the code, D.C.F is to be provided in Zone IV and V. In Zone I, the increase in this cost is not much, but there is an increase in cost for zones II and III. For the buildings considered, in Zone III the increase in cost ranges from 5% to 15% of total cost of skeletal frames and increase in quantity of steel ranges from 17% to 39% of total quanitity of steel in the skeletons. By interpolation, these ranges are 4% to 10% and 12% to 27%, respectively, for Zone II.

(ii) From the results of all the three buildings studied, it can be inferred that providing D.C.F is uneconomical in Zone I, but economical in Zones III and V. However, difference in cost between D.C.F and O.C.F(K=1.6) is not significant in Zone III. It can be seen by interpolation that providing D.C.F will be uneconomical in Zone II.

(iii) Cross sectional sizes of members greatly influence the premium for earthquake resistance in structures.

On the basis of this study it is recommended that :

(i) The code should allow only ductile detailing for Zone IV and V. It should allow relaxed ductile detailing for Zone III. Buildings in Zones I and II should be required to be designed only as per IS:456-1978 with performance factor K= 1.0.

(ii) This study has been based on three buildings all of which are framed structures. Maximum number of storeys was eight. More such systematic studies should be made on buildings with shear walls, more number of storeys, varying configuration and design features, etc.

(iii) The cost implication of providing R.C. frame versus shear wall construction in Indian conditions for different zones should also be investigated on similar lines.

APPENDIX - A : REFERENCE

1. IS:1893-1984, Indian Standard Criteria for Earthquake Resistant Design of Structures, Bureau of Indian Standards, New Delhi.

2. IS:1893-1975, Indian Standard Criteria for Earthquake Resistant Design of Structures, Bureau of Indian Standards, New Delhi.

3. IS:4326-1976, Indian Standard Code of Practice for Earthquake Resistant Design and Construction of Buildings, Bureau of Indian Standards , New Delhi.

4. IS:456-1976, Indian Standard Code of Practice for Plain and Reinforced Concrete, Bureau of Indian Standards, New Delhi.

5. Degenkolb, H. J. (Chairman), S. F. Gizienski, J.F.Meechan, D.L.Messinger and C. W. Pinkham, 1970, 'Report of the Ad Hoc Committee on Cost of Design for Earthquakes', Appendix-B, Proceedings of Structural Engineers Association of California, 171 Second Street, San Francisco, California 94105.

6. Wilson,E.L., J.P. Hollings and H.H. Dovey, 1979, Three Dimensional Analysis of Building Systems (Extended Version) - ETABS, Report No.EERC 75-13, College of Engineering, University of California, Berkeley, California,U.S.A.

7. IS:875-1964, Indian Standard Code of Practice for Structural Safety of Buildings: Loading Standards, Bureau of Indian Standards, New Delhi.

8. SP:16(S&T) 1980, Design Aids for Reinforced Concrete to IS:456-1978, Bureau of Indian Standards, New Delhi.

9. Chopra, A.K. and N.M. Newmark, 1980, 'Analysis', in Design of Earthquake Resistant Structures, E. Rosenblueth, (Ed.), Pentech Press, London.

PART III

COMPARISON

OF

SEISMIC COEFFICIENT AND RESPONSE SPECTRUM METHODS

OF THE IS CODE

1. INTRODUCTION:

IS:1893 - 1984 (Ref.1) recommends two methods, the seismic coefficient method and the response spectrum method, for determining seismic forces on multistorey buildings. With these guidelines of the code, it is expected that the level of protection to structures against earthquakes provided by the two methods is about the same at least for regular and well-behaved routine buildings. There is a need to investigate this aspect for comparision of levels of protection offered by the two methods of the code.

For response spectrum method the time periods are determined by dynamic analysis. For seismic coefficient method, the code suggests use of experimental observation on similar buildings or calculation by any rational method of analysis. In absence of such data, it suggests that fundamental time period for moment resisting frame buildings without bracing or shear walls may be taken as 0.1n, where n is the number of storeys including basement storeys. Usually a designer is not equipped with data on experimental observations or tools for any other rational method of analysis for determination of time periods and rather heavily banks on this formula. In both the methods, design seismic lateral loads depend on proper assessment of time periods. It is also of interest in this study to see if the widely used formula for time period given by the code in seismic coefficient method is comparable to actual time periods obtained by dynamic analysis for Indian buildings, and whether the two methods yield comparable lateral loads to be applied in design. This aspect has been investigated on the three

buildings described in Part II of this thesis. Dynamic analysis was done for these buildings and their time periods and design forces compared with those obtained by code formula. The institutional and residential buildings are routine type structures, while the industrial building is irregular building to the extent that it has a higher intermediate storey height. Design forces have been presented to show the difference of results obtained by the two methods. Suggestions have been given for consideration by the Earthquake Engineering Sectional Committee before bringing out future editions of the code.

2. SEISMIC COEFFICIENT VERSUS RESPONSE SPECTRUM METHODS:

The response spectrum method described by IS:1893 essentially requires dynamic analysis to evaluate seismic forces on a structure. With time periods obtained from dynamic analysis, design forces are calculated using average accelaration coefficient, $\frac{Sa}{g}$, given in average accelaration spectrum (Fig.1a).

The controlling factor in earthquake resistant design is this design spectrum. A design spectrum decides the level of seismic design force or displacement as a function of natural periods of vibration and damping characteristics of structure. According to Housner and Jennings (Ref.2):

> The level of force prescribed by a design spectrum is to be associated with specified level of material resistance, for example, the allowable design stresses or strains. The resultant effect is, thus a specification of the required earthquake resistance of a structure and its elements. If the material resistance is stated in terms of allowable stresses, the design spectrum is a

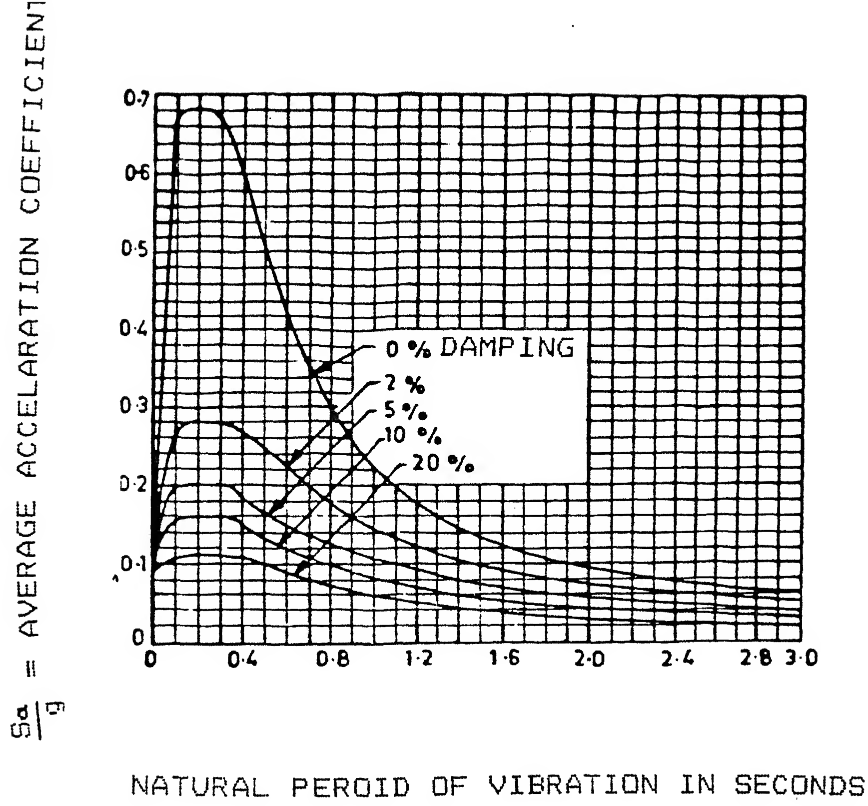


FIGURE 1(a). AVERAGE ACCELARATION SPECTRA [FROM Ref.(1)]

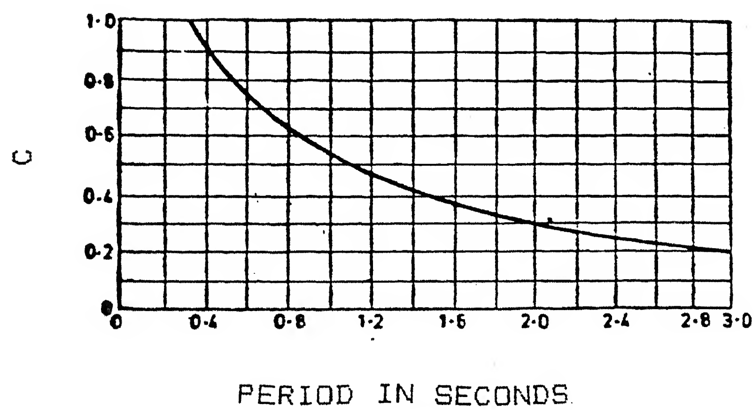


FIGURE 1(b). C VERSUS PERIOD [FROM Ref.(1)]

FIGURE 1. AVERAGE ACCELARATION SPECTRUM AND VARIATION OF C VERS

PERIOD

specification of the strength of structure and its elements; if the material resistance is expressed in terms of permissible ductile strains, the design spectrum becomes a specification of the capacity of the structure to deform, that is, the ductility it must have.

There are four major factors which combine to determine the capacity of structures to resist earthquakes (1) the level of the design spectra (2) the designated spectral damping (3) the permissible stresses and strains and (4) the method of determining the natural periods of vibration of the structures. Design criteria can be incomplete unless all four of the factors involved are specified.

If provisions of IS:1893 are observed, it is seen that an explicit specification for spectral damping for structures is lacking. However its Appendix-F suggests 5% to 10% damping for concrete structures. It does not say where to use the lower bound and where to use upper bound. Hence a designer may choose any value within this range based on his judgement. It is evident that use of 10% damping yields about 20% lower design forces. Spectral damping to be used depends on several factors and cannot easily be decided. The type of structure and materials used in construction greatly influence the damping values. Lack of clear recommendation in this regard causes ambiguity in code requirements, because based on personal judgement, same structure can be designed to have different levels of protection when designed by different persons. This ambiguity must be removed and clear specifications be given in the code for different types of structure.

If the expressions for lateral loads by the seismic coefficient method and response spectrum method are closely observed, keeping all other terms constant, the expression for storey shear in seismic coefficient method has terms $\alpha_o C$ in place of $F_o \frac{Sa}{g}$ in corresponding expression of response

spectrum method. From Table (1), ratio of F_o to α_o can be seen to be equal to 5 for all zones. The curve given in Fig.(1b) showing the variation of C with respect to time period is exactly same except for very low period as the average accelaration spectrum (Fig.1a) given by the code corresponding to 5% damping with a multiplication factor of 5. That means, $\alpha_o C$ and $F_o \frac{Sa}{g}$ will yield same values for all time periods with 5% damping. In other words, lateral forces in first mode of a structure with 5% damping, analyzed by response spectrum method, should be of the same order asthose obtained by seismic coefficient method, provided time period used in the two methods is the same. It thus appears that response spectrum method will give lower design forces especially if higher value of damping (10%) is used. Secondly even if time periods used in the two methods are same, response spectrum method will give somewhat lower value of storey shear because total mass is used in seismic coefficient method, while generalized or modal mass (participating mass) is used in dynamic analysis. However, some of this disparity is compensated by a rather conservative rule for combination of contributions from different modes as given in IS:1893.

Time period used in design is another important factor in deciding the level of earthquake forces to be applied in design. It needs to be seen for Indian buildings, as to how close arethe natural periods given by the formula T = 0.1 n, suggested by the code for seismic coefficient method, to periods calculated by dynamic analysis. There is bound to be some difference between response spectrum and seismic coefficient methods if this expression for determining the time periods is not accurate.

TABLE 1 : VALUES OF BASIC SEISMIC COEFFICIENTS AND SEISMIC ZONE FACTORS IN DIFFERENT ZONES [FROM (Ref.1)]

Sl. No.	Zone No.	Seismic Coefficient Method	Response Spectrum Method
		Basic horizontal seismic coefficient α_o	Seismic zone facror for average acceleration spectra to be used with Fig.2, F_o
(1)	(2)	(3)	(4)
i)	V	0.08	0.40
ii)	IV	0.05	0.25
iii)	III	0.04	0.20
iv)	II	0.02	0.10
v)	I	0.01	0.05

3. ANALYSIS:

All the three buildings have been analyzed by a standard three dimensional analysis computer package, ETABS. It has been briefly described in Part II. It is a special purpose building analysis program which assumes floors of the building as rigid in their own plane and for lateral load analysis treats each floor level to be having three degrees of freedom, two translational in mutually perpendicular directions and a rotational degree of freedom. The program accounts for rigid zones at ends of columns and beams. This feature has been used in analysis of buildings. Columns have been assumed to be fixed at foundation level. Design forces have been calculated by combining contributions from three significant modes in the direction under consideration, i.e., for torsionally uncoupled motion, three modes and for coupled motion, adequate number of modes such that at least three modes with predominant motion in the direction under consideration are included. The superposition of individual mode response was done using the expression given in IS:1893, combining factored contributions of both square root of sum of squares and absolute sum values. The mass of the buildings was assumed to be lumped at floor levels. Mass at each floor was calculated by summing mass within half storey height above and half storey height below the floor under consideration. The reduction in live loads was made for calculation of mass, as per IS code. Lumped mass values used in dynamic analysis for the three buildings are given in table 2. Of the three buildings considered, industrial building is fairly symmetric and had little torsion. The institutional and residential buildings have torsional coupling in

TABLE 2 : LUMPED MASS OF DIFFERENT BUILDINGS.

(a) INSTITUTIONAL BUILDING:

Level	Mass in kg.
4	500,100
3	794,000
2	794,000
1	794,000

(b) RESIDENTIAL BUILDING:

Level	Mass in kg.
8	116,000
7	168,000
6	168,000
5	168,000
4	168,000
3	168,000
2	168,000
1	168,000

(c) INDUSTRIAL BUILDING:

Level	Mass in kg.
3	709,000
2	1,011,000
1	1,650,000

longitudinal direction and the actual eccentricities have been provided for in the analysis. Modulus of elasticity for structural concrete, E_c, was calculated using the formula $E_c = 5700 \sqrt{f_{ck}}$ given in IS:456-1978 (Ref.3), where f_{ck} is the characteristic cube strength of concrete in MPa. Moment of inertia of sections was calculated for uncracked sections with overall rectangular dimensions, ignoring steel reinforcement, cracking of concrete and flange effect of slabs.

4. RESULTS AND DISCUSSION:

4.1 FOUR-STOREY INSTITUTIONAL BUILDING:

This building has no torsional coupling in the transverse direction (Y-direction) because of symmetry (Fig.1 of Part II). Table (3) gives the periods and mode shapes obtained for the building.

In the transverse direction, the fundamental period is 0.787 seconds as against 0.4 seconds given by the expression, 0.1n. Thus the period obtained is 1.97 times that given by the expression 0.1n. In the longitudinal direction, thefundamental period is 1.328 seconds, i.e., 3.32 times 0.4 seconds. The significant difference in periods of the building in the two directions is because of the unequal stiffness of columns in the two directions. Column depth in the longitudinal direction is about half of that in the transverse direction. The extraordinarily longer periods by dynamic analysis as compared to the ones given by 0.1n are discussed later .

The forces resulting from seismic coefficient and response spectrum methods are given in Table(4) and Fig.(2). In transverse direction, storey shear obtained by the response spectrum method as a percentage of that from seismic coefficient method ranges

TABLE 3 : PERIODS AND MODE SHAPES FOR THE INSTITUTIONAL BUILDING.

LEVEL		TRANSVERSE (Y-dir)			LONGITUDINAL (X-dir)				
MODE		1	2	3	1	2	3	4	5
PERIODS (Second)		0.787	0.252	0.146	1.328	0.755	0.407	0.242	0.240
4	trans.	1.00	1.00	1.00	1.00	0.12	1.00	1.00	1.00
	rot.	–	–	–	(0.001)	(1.000)	(-0.001)	(0.005)	(-0.146)
3	trans.	0.90	0.18	-0.84	0.94	0.12	0.37	-0.60	-0.56
	rot.	–	–	–	(0.001)	(0.910)	(-0.002)	(0.001)	(-0.031)
2	trans.	0.73	-0.59	-0.41	0.79	-0.12	-0.54	-0.67	-0.70
	rot.	–	–	–	(0.001)	(0.730)	(-0.002)	(-0.003)	(0.086)
1	trans.	0.46	-0.79	0.91	0.58	-0.12	-0.95	0.81	0.79
	rot.	–	–	–	(0.001)	(0.476)	(-0.002)	(-0.004)	(0.119)

* Figures in the parantheses are rotations in radians.

TABLE 4 : FORCES RESULTING FROM DYNAMIC AND STATIC ANALYSES FOR THE INSTITUTIONAL BUILDING.

	Storey Shear (kN)					Storey Moment (kN-m)				
		Dynamic X		Dynamic Y			Dynamic X		Dynamic Y	
Level	Static Force	Force	% of St.	Force	% of St.	Static Moment	Moment	% of St.	Moment	% of St.
4	2120	660	31	1030	49	7970	2730	34	3860	48
3	4130	1560	38	2080	50	23480	8580	37	11460	49
2	5110	2110	41	2770	54	38990	16260	42	21440	55
1	5430	2600	48	3390	62	69790	28140	40	36340	52

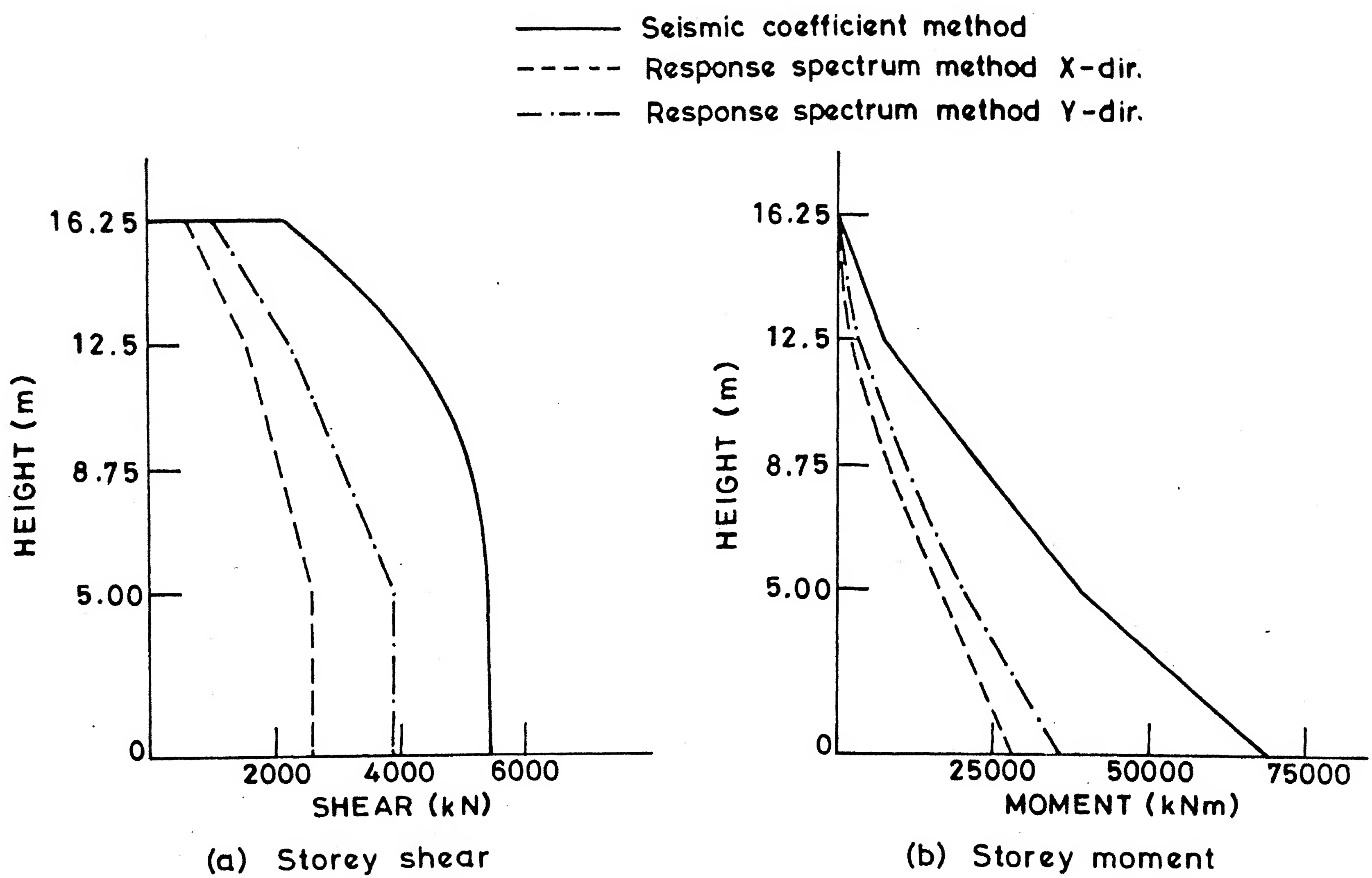


FIGURE 2. COMPARISON OF FORCES RESULTING FROM RESPONSE SPECTRUM AND SEISMIC COEFFIENT METHODS FOR THE INSTITUIONAL BUILDING

from 62 at level 1 to 49 at roof level. Corresponding range for storey moments is from 52 to 48 percent. If time period of 0.787 which is obtained by dynamic analysis is used (in place of 0.4 seconds) in seismic coefficient method, these ranges are 92 percent to 72 percent.

As discussed earlier, in the longitudinal direction, the discrepancy in time periods is much more, leading to further reduction in design forces. In this direction, the response spectrum method gives storey shears in the rangeof48 percent (at level 1) to 32 percent (at roof level) of the storey shears obtained by seismic coefficient method. Corresponding range for storey moments is from 40 percent at (level 1) to 34 percent (at roof level). If time period obtained by dynamic analysis is used in seismic coefficient method, these values change to 99 percent to 63 percent for storey shear and 80 percent to68 percent for storey moment.

4.2 EIGHT-STOREY RESIDENTIAL BUILDING:

This building has no torsional coupling in its transverse direction because of symmetry in that direction, while in the longitudinal direction, its motion is coupled. Its natural periods and mode shapes are given in Table (5).

The fundamental time period in transverse direction is 1.365 seconds which is about 1.71 times 0.8 seconds obtained by using formula 0.1n. In longitudinal direction the time period is 1.185 seconds, which is 1.48 times that estimated in accordance with 0.1n.

Storey shears and moments obtained by the response spectrum and seismic coefficient methods are given in Table (6) and Fig.(3). In transverse direction (i.e., Y-direction), response spectrum method yields storey shears which are 66 percent (at

TABLE 5 : PERIODS AND MODE SHAPES FOR THE RESIDENTIAL BUILDING. .

LEVEL		TRANSVERSE (Y-dir)			LONGITUDINAL (X-dir)			
MODE		1	2	3	1	2	3	4
PERIODS (second)		1.365	0.437	0.236	1.185	1.043	0.378	0.334
8	trans.	1.00	1.00	1.00	1.00	1.00	1.00	1.00
	rot.	–	–	–	(−0.053)	(0.449)	(−0.057)	(0.437)
7	trans.	0.95	0.70	0.28	0.96	0.96	0.73	0.72
	rot.	–	–	–	(−0.051)	(0.431)	(−0.041)	(0.308)
6	trans.	0.89	0.30	−0.46	0.90	0.90	0.32	0.297
	rot.	–	–	–	(−0.048)	(0.403)	(−0.019)	(0.133)
5	trans.	0.80	−0.17	−0.86	0.81	0.81	−0.16	−0.20
	rot.	–	–	–	(−0.044)	(0.367)	(0.008)	(−0.070)
4	trans.	0.69	−0.58	−0.63	0.71	0.70	−0.60	−0.63
	rot.	–	–	–	(−0.038)	(0.319)	(0.031)	(−0.252)
3	trans.	0.57	−0.85	0.05	0.58	0.57	−0.87	−0.88
	rot.	–	–	–	(−0.032)	(0.264)	(0.048)	(−0.372)
2	trans.	0.44	−0.90	0.70	0.44	0.42	−0.92	−0.89
	rot.	–	–	–	(−0.025)	(0.203)	(0.043)	(−0.393)
1	trans.	0.29	−0.71	0.84	0.29	0.25	−0.71	−0.64
	rot.	–	–	–	(−0.017)	(0.131)	(0.042)	(−0.304)

TABLE 6 : FORCES RESULTING FROM DYNAMIC AND STATIC ANALYSIS FOR THE RESIDENTIAL BUILDING.

	Storey Shear (kN)					Storey Moment (kN-m)				
		Dynamic X		Dynamic Y			Dynamic X		Dynamic Y	
Level	Static Force	Force	% of St.	Force	% of St.	Static Moment	Moment	% of St.	Moment	% of St.
8	340	190	56	190	56	1070	590	55	600	56
7	720	420	58	410	57	3040	1950	64	1980	65
6	1010	620	61	580	57	5920	3910	66	3640	62
5	1220	760	62	660	54	9460	6040	64	5650	60
4	1360	850	63	750	55	13440	8530	63	8150	61
3	1450	910	63	790	54	17690	11300	64	10100	57
2	1490	1040	70	890	60	22080	14180	64	12510	57
1	1510	1140	75	990	66	29140	18700	64	16350	56

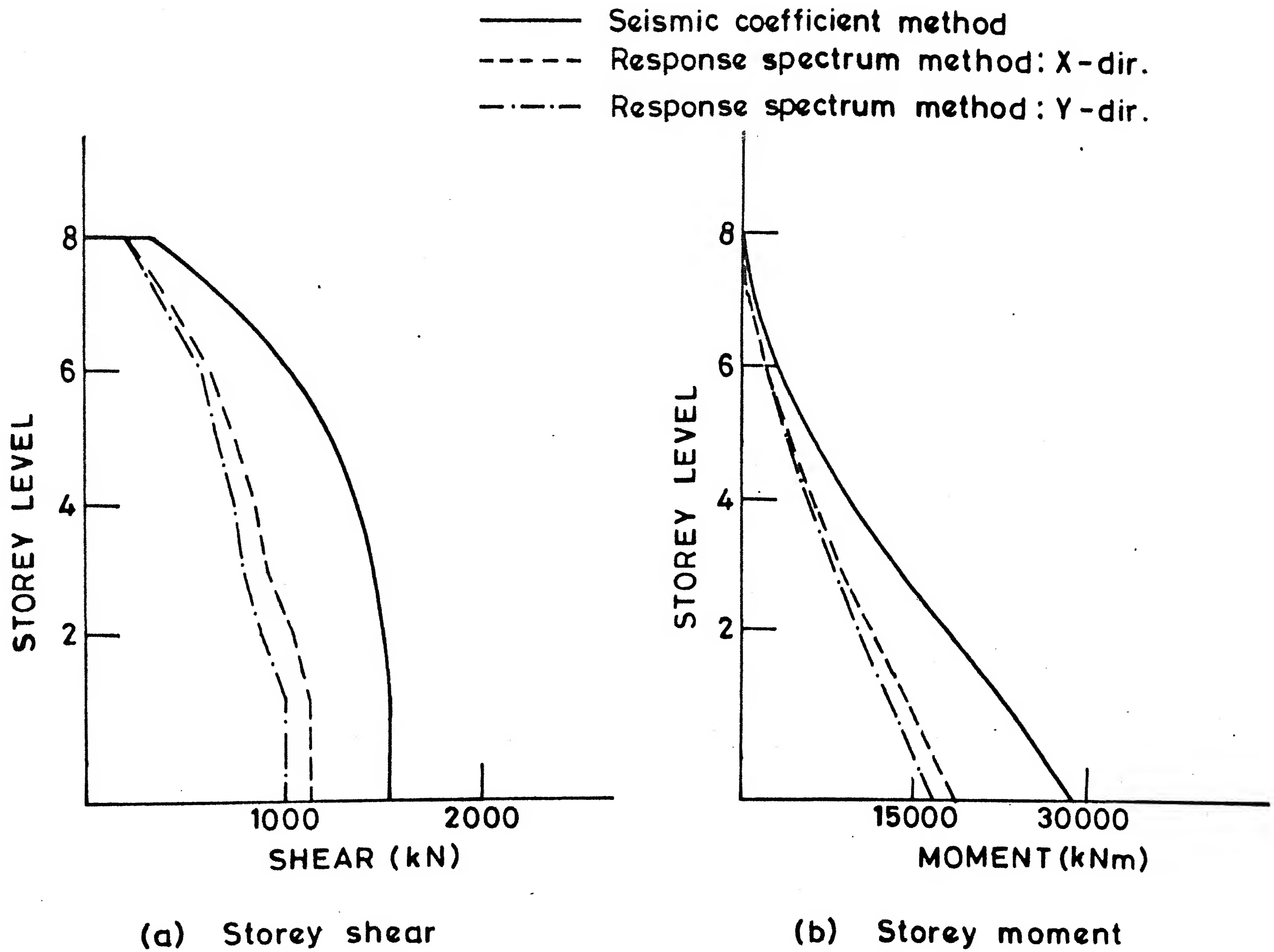


FIGURE 3. COMPARISON OF FORCES RESULTING FROM RESPONSE SPECTRUM AND SEISMIC COEFFIENT METHODS FOR THE RESIDENTIAL BUILDING

TABLE 7 : PERIODS AND MODE SHAPES FOR THE INDUSTRIAL BUILDING.

LEVEL	TRANSVERSE (Y-dir)			LONGITUDINAL (X-dir)		
MODE	1	2	3	1	2	3
PERIODS (Second)	0.914	0.331	0.168	0.993	0.367	0.179
3	1.00	1.00	1.00	1.00	1.00	1.00
2	0.89	0.28	0.85	0.90	0.37	-0.83
1	0.25	-2.31	-0.13	0.26	-2.44	0.10

ranging between 2.7 m and 3.6 m, but it can still be used in exceptional cases where one or two - storey heightsare upto 5 m. This building has a storey height of 6.15 m which does not fall in the specified range.

The forces resulting from seismic coefficient and response spectrum methods are given in Table (8) and Fig.(4). In the longitudinal direction, the storey shears calculated by response spectrum method as compared to those by seismic coefficient method are 59 percent at base and 38 percent at roof. Corresponding values of storey moments are 40 percent and 38 percent, respectively.

In transverse direction response spectrum method results in storey shears which are 63 percent at base and 42 percent at roof level of those by seismic coefficient method. Similarly, corresponding values for storey moments are 43 percent and 42 percent respectively.

5. DISCUSSION AND CONCLUSIONS:

In this study, the level of protection provided by IS Code provisions for the two methods specified (i.e.,response spectrum and seismic coefficient methods) is compared. The difference in earthquake forces mainly depends on the time periods and damping values used in the design. IS:1893-1984 recommends design spectrum which gives values of average accelaration for different time periods and different damping. The Code also suggests use of 5 percent to 10 percent of damping for concrete structures, but is not specific about the value to be used. Choice of damping value will greatly influence design forces in the response spectrum method.For seismic coefficient method the code suggests an approximate formula for time period as 0.1n.

TABLE 8 : FORCES RESULTING FROM DYNAMIC AND STATIC ANALYSES FOR THE INDUSTRIAL BUILDING.

Level	Storey Shear (kN)					Storey Moment (kN-m)				
	Static	Dynamic X		Dynamic Y		Static	Dynamic X		Dynamic Y	
		force	% of static	force	% of static		force	% of static	force	% of st
3	3110	1190	38	1300	42	10880	4160	38	4560	42
2	5610	2460	44	2640	47	45410	19250	42	20290	45
1	6340	3730	59	3970	63	73960	29390	40	31920	43

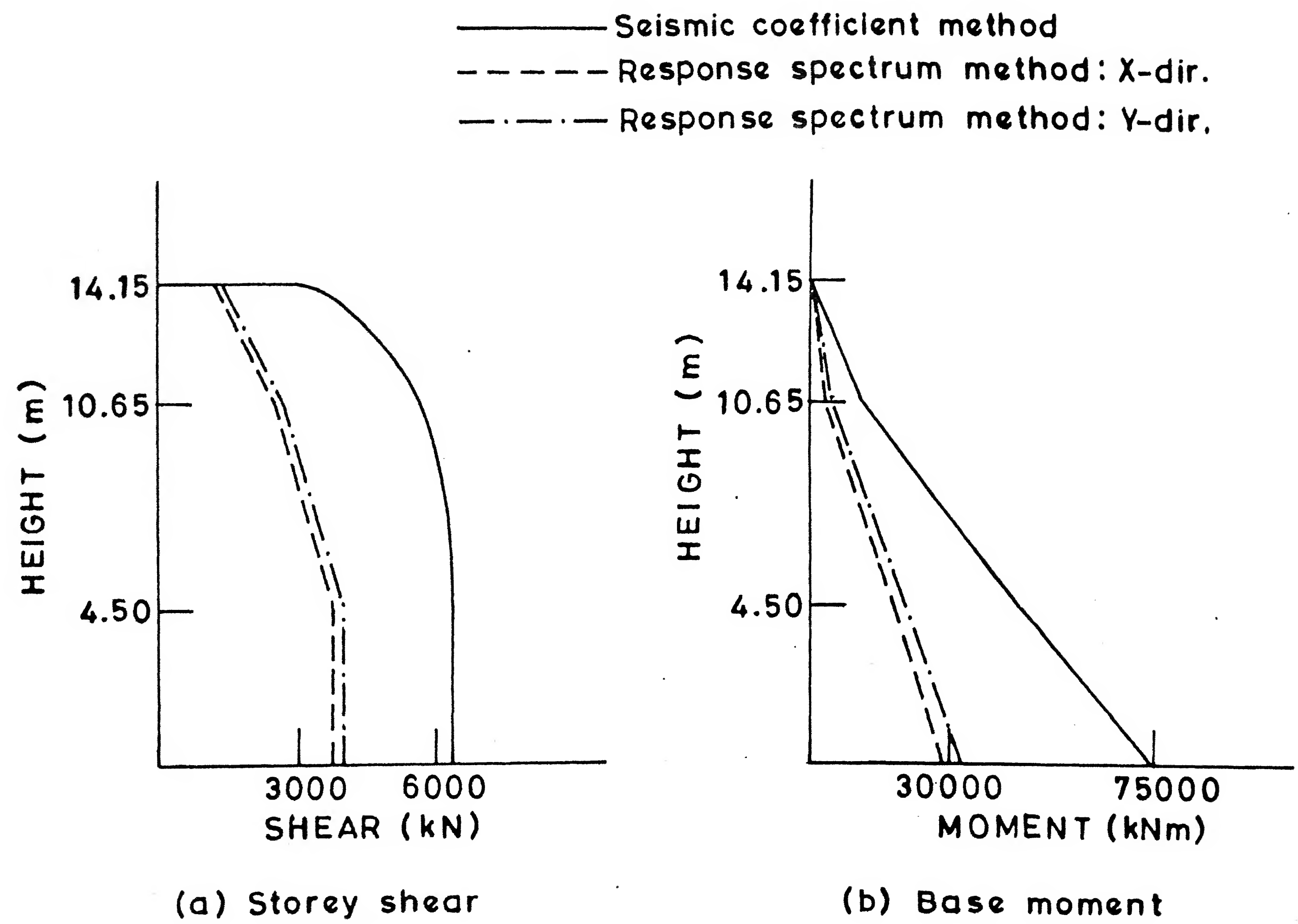


FIGURE 4. COMPARISON OF FORCES RESULTING FROM RESPONSE SPECTRUM AND SEISMIC COEFFIENT METHODS FOR THE INDUSTRIAL BUILDING

Three buildings have been analyzed by response spectrum and seismic coefficient methods. The study revealed that the formula given by IS Code for fundamental period as 0.1n highly underestimates actual periods for Indian buildings. In fact, this formula for time periods has originated from California, U.S.A. There, the structures are more stiff, because the building codes require shearwalls to be provided in buildings. Further, structures built in U.S.A. are not as massive as those built in India. Their partitions and claddings are lighter as compared to usual brick partitions used our country. Hence, the American systems are stiffer and less massive, which results in lesser time periods. Buildings in our country are not governed by any stringent requirement by codes to provide shear walls or bracings. This results in more flexible systems. Partition walls are usually of brick work. Also building floors and floor finishes used here, are thicker. In effect these factors tend to increase time periods of the building.

The study also revealed that the forces calculated by response spectrum and seismic coefficient methods are very much different. This is because of the fact that in seismic coefficient method time periods used are based on the formula 0.1n which gives much lower periods for Indian buildings. Design forces depend on time periods in both methods. While in response spectrum method, actual time periods are used, in seismic coefficient method lower values of time periods are used because of this formula. This results in higher design forces if calculations are done by the latter method. For the three buildings studied, with damping value of 5 percent, the storey shears obtained by response spectrum method are in the range of 48 percent to 75 percent at base to 31 percent to 56 percent at roof

of those obtained by seismic coefficient method. If 10 percent damping were to be used, this disparity would have further increased. Hence the protection provided by the two methods are significantly different. However, if realistic periods as obtained by dynamic analysis and used in seismic coefficient method alongwith 5 percent damping, design forces by the two methods are fairly close.

For a building having brick in-fill walls, there could be structural contribution from in-fills, however there is no consensus as yet on treating these as integral partof the structure. Brick in-fills provide increased stiffness in the initial stages of loading, and once the forces cross a certain limit, walls will separate out from frames and then frames will come into action in resisting the forces. This interaction of brick in-fills with frames structurally, as an integral part, is not yet fully understood. If in-fill walls are treated as non-structural, the periods obtained are very large as compared to the code formula of 0.1n.

From this study, following conclusions can be drawn.

(1) The response spectrum and seismic coefficient methods do not provide same levels of protection to buildings. However, both methods must provide comparable design forces for regular structures. Thus, these provisions need to be rationalized.

(2) The code should be very specific about what values of damping are to be used, so that the level of protection against earthquakes provided in a structure will have a common basis, instead of designer's discretion.

(3) Earthquake Engineering Sectional Committee should review

by 0.1n and suggest

a more realistic expression.

(4) Proper consideration is to be given to include the effect of brick in-fills in assessing dynamic characteristics of buildings, because this will further change the time periods and hence the design forces.It will help to carry out extensive ambient and forced vibration tests on multi-storeybuilding to obtain better understanding of the dynamic characteristics of the Indian buildings. Even though these tests are for low level vibrations, they will give fairly good indication of actual periods of building for initial stages of loading.

(5) Modal combination rule given by IS Code is very conservative. However, this compensates the effect of using modal mass in place of actual mass in dynamic analysis. Instead of compensating this way , the Earthquake Engineering Sectional Committee may consider increasing the average accelaration spectrum and adopting square root of sum of squares relation for modal combination.

(6) This study is based only on three buildings. All the buildings are framed structures with a maximum of eight storeys. Similar studies should be undertaken for framed structures with shear walls, for different configurations, more number of storeys, etc.

APPENDIX - A : REFERENCES

1. IS:1894-1984, Indian Standard Criteria for Earthquake Resistant Design of Structures, Bureau of Indian Standards, New Delhi.

2. Housner, G.W. and P.C. Jennings, 1982, Earthquake Design Criteria, Earthquake Engineering Research Institute, 2620 Telegraph Avenue, Berkeley, California, 94704.

3. IS:456-1978, Indian Standard Code of Practice for Plain and Reinforced Concrete, Bureau of Indian Standards , New Delhi.

APPENDIX - B : NOTATIONS

α_o - basic horizontal seismic coefficient in IS:1893

C - a coefficient defining flexibility of structure in IS:1893

F_o - seisimic zone factor in IS:1893

$\frac{Sa}{g}$ - averave accelaration coefficient given in IS:1893

n - number of storeys including basement storey